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# THE PRACTICAL DESIGN OF IRRIGATION WORKS

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BY

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# PREFACE.

THIS book is written with the special object of setting forth the principles governing the design of irrigation works in a detailed, as well as a general aspect, so that any engineer, even if not specially conversant with this branch of engineering science, may be enabled, by the use of proper dis-

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left without guide in the selection of suitable types, and consequently is unconsciously led to follow old precedents in design, to the exclusion of more modern improvements, or to the exercise of his own ingenuity.

The first four chapters deal with the sections of various parts of irrigation structures, together with their disposition. This groundwork is most essential in view of the errors perpetrated in many designs in these comparatively elementary, but none the less important matters, some of which have been brought to notice in this work. The chapter on Weirs is believed to be the only literature on this subject, considered apart from that of Dams. The fifth chapter deals with Hydraulic Formulas and Tables

## ERRATA.

CHAPTER V. —Page 153, in second formula from top, "$cl\frac{2}{3}\sqrt{2g}\,(h^{\frac{3}{2}} - h^{\frac{3}{2}})$" *should read* "$cl\frac{2}{3}\sqrt{2g}\,(h_2^{\frac{3}{2}} - h_1^{\frac{3}{2}})$."

In formula 2a, "$h_1\sqrt{h}$" *should be* "$h_1\sqrt{h_1}$."

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In order to effect this purpose to the best advantage, an entirely new departure has been adopted in the method of instruction pursued. This consists mainly in the production of plans of existing or projected works, combined with a systematic analysis of the salient points involved in each design. This critical examination will expose the good and the bad features, so that the reader will be enabled to exercise discrimination in what to accept and follow and what to reject. At the same time, in order to thoroughly illustrate the subject, alternative designs for the works thus reviewed, embodying the suggested improvements, are provided, wherever such appears desirable.

The method of procedure thus sketched is clearly superior, from an instructive point of view, to the ordinary one of dumping down a number of record plans of various works, some of which are of quite obsolete form, without any critical remarks on their merits or demerits from the standpoint of modern development. The inexperienced designer in that case is left without guide in the selection of suitable types, and consequently is unconsciously led to follow old precedents in design, to the exclusion of more modern improvements, or to the exercise of his own ingenuity.

The first four chapters deal with the sections of various parts of irrigation structures, together with their disposition. This groundwork is most essential in view of the errors perpetrated in many designs in these comparatively elementary, but none the less important matters, some of which have been brought to notice in this work. The chapter on Weirs is believed to be the only literature on this subject, considered apart from that of Dams. The fifth chapter deals with Hydraulic Formulas and Tables and their practical application, and includes several discharge tables never before published, based on a new and original theory, which have been added with a view to facilitate the calculations of the numerous problems and examples subsequently introduced. The remaining chapters deal with works as a whole, and consist of the practical application of the results arrived at in the previous five chapters, together with the elucidation of such new problems as may arise. Among these, by far the most important is that of the application of the new theory of the so termed "frictional stability" of a sand substratum, in a practical and conclusive manner which has never previously been attempted. With the sole exception of "High Dams," graphical solutions have invariably been adopted.

W. G. BLIGH.

*July*, 1907.

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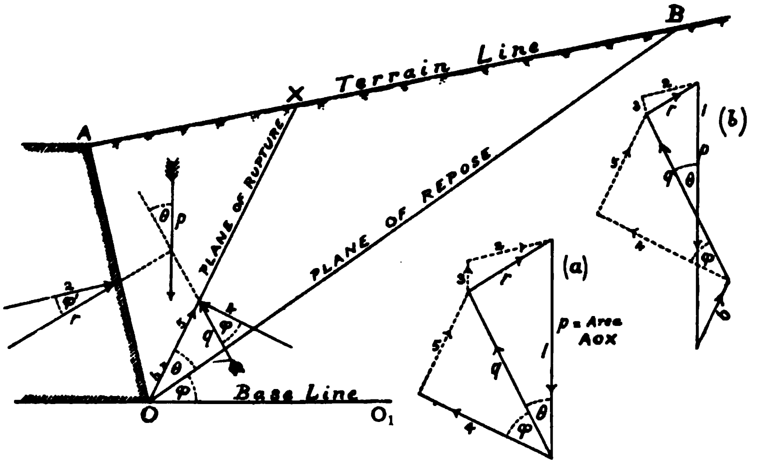
# THE PRACTICAL DESIGN OF IRRIGATION WORKS.

## CHAPTER I.

### RETAINING WALLS.

(1) Retaining walls, whose function is the retaining or holding back of earthen banks, form one of the most considerable items of the component parts of masonry structures. The design of such should satisfy conditions comprising the utmost economy of material in combination with absolute efficiency. To effect this desirable end with any degree of accuracy and in accordance with scientific principles, it will be necessary to possess a thorough understanding of the theoretical problem involved, without which the designer has to fall back upon empirical rules of doubtful value.



Figs. 1, 1a, 1b.

(2) The diagram in Fig. 1 presents a graphical statement of the theory of earth pressure. $AO$ is the back of the retaining wall, $AB$ the terrain line, or top surface of the bank which is upheld, and $OO_1$ the horizontal base

line representing the natural surface of solid ground. *OB* is the plane of repose of the earth forming this backing, the angle $BOO_1$, or $\phi$, being thus the angle of repose, *i.e.*, the steepest slope at which the earth will stand when unsupported. The triangle *AOB* represents, therefore, the largest prism of earth held up by the wall.

In case of failure of the wall, the earth backing will break away in some line *OX* in the prism *AOB*, intermediate between *AO* and *OB*, and *OX* is termed the plane of rupture. Of all possible positions of *OX*, that one which ensures the maximum thrust on the wall has alone to be considered as the true plane of rupture, and thus the prism *AOX* is the portion of the earth backing tending to overturn the wall, and will be found, as hereafter explained, to have a much greater effect than the larger whole prism *AOB*, on the supposition that the plane of rupture is coincident with that of repose, which is not the case.

(3) By reference to Figs. 1a and 1b, the forces in equilibrium will be found as follows:—

(1) The weight of the prism *AOX* acting vertically.

(2) The normal reaction of the retaining wall acting at right angles to the direction of the back of the wall *OA*.

(3) The friction of the prism on the inner surface of the retaining wall acting upwards.

(4) The normal reaction of the plane of rupture *OX*.

(5) The friction of the prism *AOX* on the plane of rupture acting upwards.

(6) The cohesion of the earth in the plane of rupture acting upwards.

(4) Fig. 1a represents the force polygon of forces 1 to 5; Fig. 1b the same, but inclusive of (6). In both diagrams forces 2 and 3 are components of $r$, and 4 and 5 of $q$. The polygon is thus resolved into the triangle of forces $p$ $q$ $r$. Of these $p$ is the weight of the prism *AOX*, represented by the area of the triangle *AOX*; $q$ is the reaction of the plane of rupture, and $r$ the resultant pressure on the back of the wall, both $q$ and $r$ being modified in direction by angles of friction. The direction of $q$ is determined by the angle $\theta$, which it forms with the vertical $P$; the length of $r$ is dependent on the value of this angle $\theta$. Now $\theta$ is also the angle between the planes of repose and of rupture, or the divergence of *OB* and *OX*. Hence the primary necessity of ascertaining the true value of this angle, on which that of the force $r$ is dependent.

(5) The two diagrams, 1a and 1b, shows clearly the great reducing influence exerted on the value of $r$ by the introduction of the comparatively small force (6) representing cohesion. No definite value can possibly be assigned to cohesion, as it must necessarily vary with differing soils and conditions of moisture, consolidation, etc.; for instance, from pure sand, which is almost devoid of the property of cohesion, to stiff clay in which this property is highly developed. Consequently we are compelled to omit

this force, though always present in a greater or less degree, from inclusion in the graphical statement.

Although this omission will apparently somewhat vitiate the value of the theory of earth pressure, as it presupposes a state of things in the condition of the earth which does not actually exist, yet it will be found that the neglect of cohesion will not preclude the successful use of the remaining five forces to which approximately correct values can be assigned, provided that the unknown value of cohesion be considered as supplying a definite, though not precisely ascertainable, factor of safety in favour of the stability of the wall.

It is well known that retaining walls, which according to received theory are designed of too weak a section to be safe, are in reality possessed of a large factor of safety. This point was fully brought out in the discussion on retaining walls in the "Minutes, Proceedings of the Institution of Civil Engineers," Vol. LXV. Consequently the theory of earth pressure, though not reducible to absolute exactitude as in the case of fluid pressure, still will be found an indispensable and reliable guide in the economical design of retaining walls. The sections need only be designed of sufficient statical properties to withstand the theoretical earth pressure. That is to say, the moment of stability of the wall need be only just equal to that of the aggressive forces, or in equivalent graphical terms, the resultant centre of pressure of the combined forces can be allowed to pass just within the outer toe of the base of the section; leaving no factor of safety beyond that supplied by the unknown influence of cohesion, and, further, designs based on a slight modification of this principle will be found in agreement with commonly received and established practice.

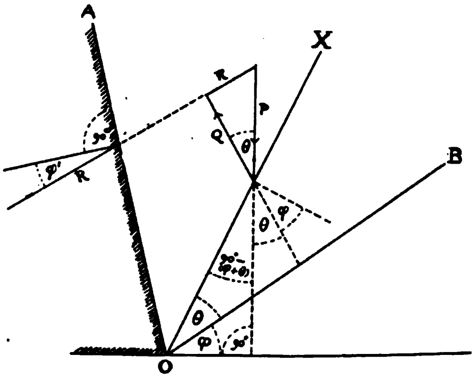


FIG. 2.

(6) Fig. 2 illustrates more fully the triangle of forces $p$ $q$ $r$ with their angles of divergence. The resultant $r$, it will be seen, forms an angle with the normal to $OA$ of the assumed value of $\phi_1$, tan. $\phi_1$ being the co-efficient of friction of earth against the wall. This force, which acts in favour of the wall, causing a downward deflection in the direction of $r$, is neglected by some theorists, but without tangible reason. It is evident that friction must develop on this surface, as well as on the plane of rupture, on any movement of the wedge of earth $AOX$; consequently it cannot be ignored. The value of $\phi_1$ is, however, not considered as equal to $\phi$, and in Chalmers' "Graphical Determination of the Stresses in the Structures of Engineering," an authority on such subjects, the value of $\phi_1$ is assessed at one half that of $\phi$. This proportional value will be adopted. As regards $\phi$, a consensus of authorities place

the slope of repose at 1½ to 1, which is what ordinary soil naturally assumes. Some clays, however, have a much flatter slope of repose, as 2 or 3 to 1; in such cases, however, the backing should be formed of prepared material, which will not have a flatter angle of repose than 1½ to 1. Sir Benjamin Baker, in the paper on the pressure of earth on retaining walls in the "Minutes of the Proceedings of the Institution of Civil Engineers," Vol. LXV., assumes 1½ to 1 as the slope of repose, although dealing with London clay with a natural angle of 3 to 1.

**(7)** The earth backing is thus theoretically considered as a fluid, having the specific gravity of earth, but subject to modification by the influence of friction on the surfaces on which it slides. The reactions of the back of the wall and of the plane of rupture, in accordance with the properties of fluids, must be normal to those surfaces, but the action of friction causes obliquity in the directions of $q$ and of $r$ from the lines normal to $OA$ and $OX$.

**(8)** In all subsequent investigations the value of $\phi$ will be assumed at 1½ to 1, $\phi_1$ at 3 to 1. With regard to $p$, the weight of earth will be taken as 1 cwt. per cubic foot, or specific gravity 1·8. Some clays weigh as much as 120 lbs. or 125 lbs. per cubic foot, or have a specific gravity of 1·9 or 2·0, but they are exceptional and can be met by special design. In hot countries, where alone irrigation is practised, the heavy clays common in Europe, which are due to glacial action, are rarely met with. Most soils do not weigh over 100 lbs. per cubic foot.

### *Position of the Plane of Rupture.*

**(9)** It has already been observed (*vide* par. 2) that the position of $OX$ is somewhere intermediate between $OB$ and $OA$. By reference to Fig. 2, it will be seen that if the plane of rupture be assumed to coincide with that of repose, the triangle $AOX$ will be coincident with $AOB$, and consequently $p$ will be at a maximum value. The angle $\theta$, however, disappears, and with it $r$. On the other hand, were $OX$ coincident with $OA$, $p$ would disappear. Some intermediate position of $OX$ must be found which will satisfy the condition of giving a greater value to $r$ than any other possible one. In works where this subject is treated by analytical methods, a formula is given whereby the angle $\theta$ can be found by calculation, but it is so involved as to be useless for practical purposes. In Chalmers' work the problem is solved by graphical process, but except in the case of a horizontal terrain line, the system adopted is too abstruse and complicated to be of practical value, though doubtless suitable for students as an exercise of ingenuity. The process adopted in this work for finding the correct value of $\theta$, where the terrain line is not horizontal, is experimental in method and as simple as possible. Various positions of the plane of rupture are tentatively assumed, the value of $r$ deduced in each case, and that inclination giving the greatest value to $r$ is adopted as the correct value of $\theta$.

**(10)** When the terrain line $AB$ is horizontal and the angle of friction is neglected, $OX$ will bisect the angle $AOB$. The proof of this can be found in any work on the subject. When the friction on $AO$ is taken into account, the direction of $r$ will be more inclined, and thus will tend to slightly modify the position of $OX$, but to so small an extent that for all practical purposes it may be entirely neglected.

(**11**) In cases where the terrain line $AB$ is inclined upwards or downwards, as shown in Figs. 3 and 4, the position of $OX$ has to be found by a tentative process as exhibited below.

Fig. 3 is a case with a terrain line inclined upwards at a slope of 2 to 1. On $AB$ any number of points, $X_1$, $X_2$, etc., are taken; these joined by the dotted lines with $O$, form so many planes of rupture, each having different values of $\theta$, viz., the angles $X_1OB$, $X_2OB$, etc. We have now to find which of these directions will ensure the greatest value of $R$, the resultant pressure on the wall. As the triangles $AOX_1$, $AOX_2$, etc., have the common base $AO$, their areas are directly proportional to their several sides, $AX_1$, $AX_2$, etc., consequently these sides can be taken as representative of their areas, *i.e.*, of $p_1$, $p_2$, etc., the weights of the prisms held up by the wall.

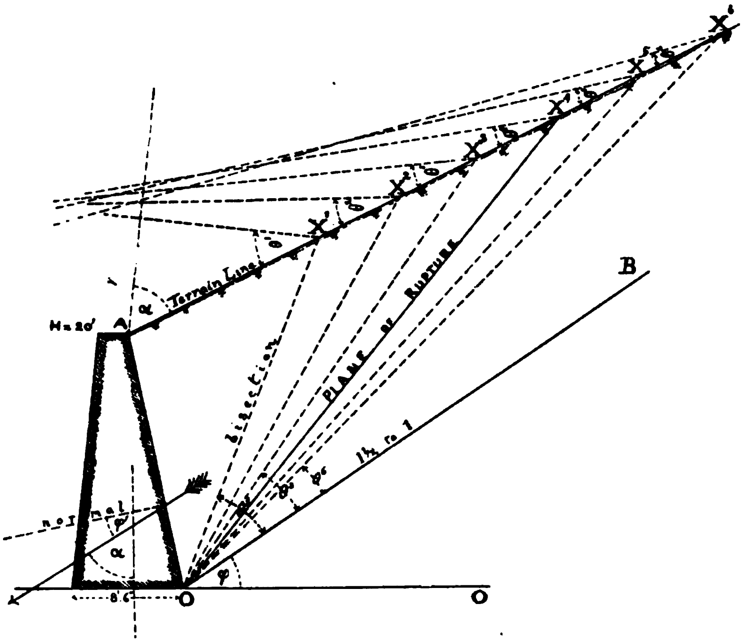


FIG. 3.

To avoid constructing a separate figure the force polygon can be joined on to the diagram by assuming $AX$ as the vertical load line. At the point $A$ the direction of $r$ is set off, making an angle with $p$ equal $\alpha$ on the figure, in the same way at the points $X_1$, $X_2$, etc., the angles $\theta_1$, $\theta_2$, which $OX_1$ and $OX_2$ make with $OB$ are set off toward the line $r$. These lines, which represent the oblique reactions of the planes of rupture, cut $r$ in several places. These several intercepts of the line above the point $A$ give the value of $r$ for each trial plane of rupture, and that angle effecting the largest intercept is to be adopted as the correct value of $\theta$. In the case in question the line 4 gives that greatest value to $r$. $OX_4$ is then the correct plane of rupture, and $X_4OB$ the correct value of the angle $\theta$.

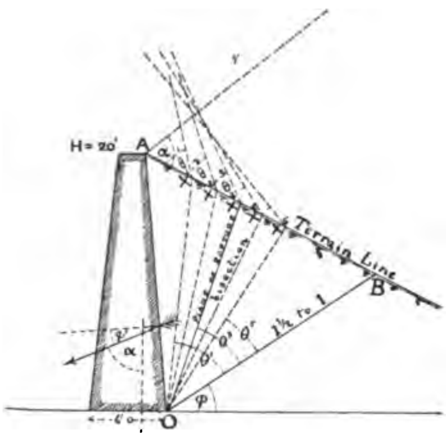


FIG. 4.

(12) In Fig. 4, where the terrain line slopes downwards, the process of finding the plane of rupture is identical with that already described. In this example the intercept of $\theta_8$ is the largest, consequently $OX_8$ is the true plane of rupture. It will be noticed that with a rising terrain, as in Fig. 3, the position of the plane of rupture lies beyond the bisection of the angle $AOX$, whereas with a falling terrain line in Fig. 4 it lies within the bisection.

(13) A third case is illustrated in Fig. 5.

Here the terrain line is horizontal, but above this is superimposed a depth of water. This is an example of the common case of a masonry weir wall with earth backing and water passing over the crest. It is clear the presence of the water will not modify the position of $OX$, which will bisect the angle $AOB$ (*vide* par. 10). For purposes of simplification the area of the water rectangle $aX$ has to be reduced to the corresponding weight of earth. Taking the specific gravity of the earth backing as 1·8, if the depth $Aa$ of the water rectangle be divided by 1·8 and $AA_1$ made equal to this, the area of the smaller rectangle $A_1X$ will then represent the corresponding weight of earth. In the example before us the depth of water $Aa$ is 4 ft. $AA_1$ is then scaled off $= \frac{4}{1·8} = 2·22$, or this division can better be effected by graphical process. Make the horizontal $Ac = 1·8$ on any scale, and $Ad = 1$, join $cd$, mark off $Ae = Aa$ and draw $eA_1$ parallel to $cd$, $AA_1$ will then be $\frac{aA}{1·8}$. The horizontal through $A_1$ should be drawn, meeting $OX$ continued, at $X_1$; the enlarged triangle $A_1OX_1$ will then represent the

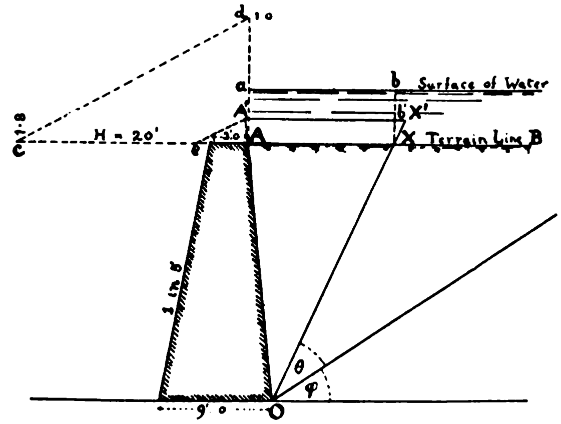


FIG. 5.

combined weight of earth and water acting on the wall, as if composed of the former material.

The slight excess in the area of the superimposed parallelopiped $AX_1$ which is somewhat larger than the rectangle $A_1X$, is too trifling for consideration, though capable of exact adjustment by further lowering the line $A_1b_1$.

(**14**) Process of drawing resultant line of pressure to base of wall profile.

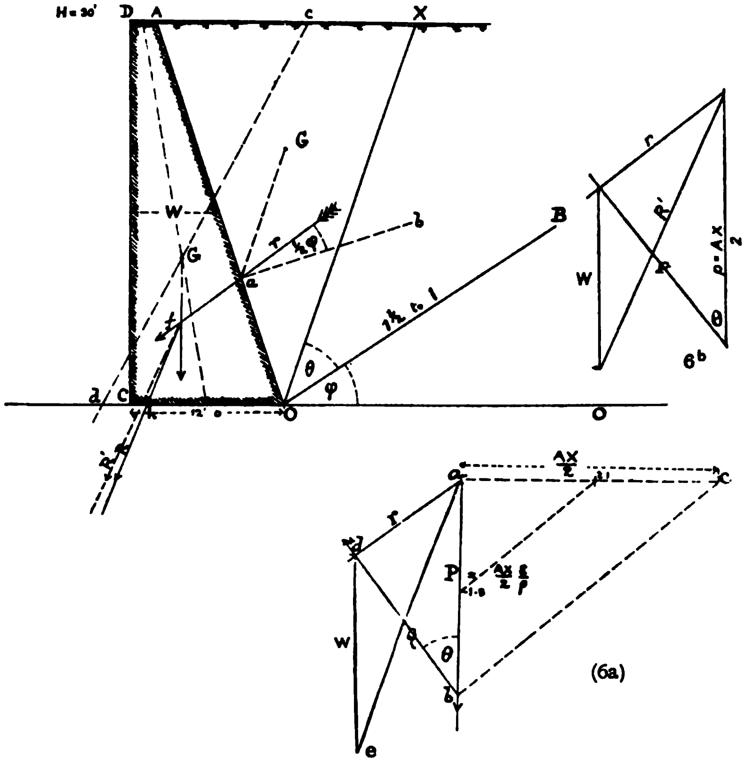
Fig. 6 represents a trapezoidal section of wall 30 feet high, 2 feet wide at crest, and 12 feet at base. The face is vertical. The object of drawing the resultant line of pressure through the base of the wall, is that the position of the point where this intersects the base forms the criterion of the stability of the section adopted. If it falls at the outer toe, the section is of just sufficient statical stability; if outside, the wall will overturn. The further the point of intersection falls within the outer toe, the greater the stability. Excess, however, would lead to waste of material. As already noticed in par. 5 the unknown force of cohesion forms a large but indefinite factor of safety. It is, however, deemed advisable to ensure a certain small additional precise factor by designing the section so that the resultant line of pressure will intersect the base within the outer toe at a distance having a certain arbitrary proportion to the width of base. This will obviate any remote possibility of crushing the material at the edge of the base, which would be liable to occur were the maximum point of pressure concentrated at this point. This inset being a fraction of the base will ensure a uniform system of design applicable to all sections and one which is in agreement with practical examples of known stability, the weight of these trapezoidal walls thus bearing a constant ratio to the base width, the top being a fixed dimension.

This fraction will be fixed at one-eighth the width of base, and has been decided on with due regard to assimilation of design to the proportions of retaining walls such as are commonly followed in practice. The problem of design now consists in adjusting the base width of the sections so that the resultant line of pressure will cut the base at a point $\frac{b}{8}$ within the toe. This can only be effected by a tentative process of trial and error. As the section is determined entirely by the base width, any alteration in the latter affects the back slope of the wall, and with it the oblique direction of $r$, which is at a fixed angle from the normal to the back, furthermore the area of the triangle of earth pressure, the position of $OX$ and value of $\theta$, are all modified by any alteration in the base width of the profile of the wall.

To express this in analytical terms would necessitate the use of formulas of so complicated a form, as to prove too cumbersome by far for practical purposes, and the time spent in working them out would greatly exceed that employed in making the graphical solution, besides the increased liability to error.

(**15**) As already noted in par. 8 the specific gravity of earth backing is taken as 1·8 for these investigations. With regard to the walls, carefully built rubble masonry will weigh from 140 lbs. to 145 lbs. per cubic foot, *i.e.*, have a specific gravity of 2¼. This value is adopted in "Water Works Engineering" as applicable to masonry dams, where it is stated that the specific gravity of the stone used with careful construction should in cases of walls subject to water pressure, bring the weight of the built masonry to not less than 145 lbs. per cubic foot.

For retaining walls of ordinary construction a specific gravity of 2¼ is deemed too high, and in this work 2·1 will be adopted for earth-retaining walls and 2·25 for stone dams and weirs.



FIGS. 6, 6a, 6b.

Granite masonry has a specific gravity of 2½, or even 3 or more, with Portland cement mortar. Brickwork varies from 100 lbs. to 120 lbs. per cubic foot, *i.e.*, from specific gravities 1·6 to 1·9. In these investigations 1·8 will be adopted as the specific gravity of brickwork. The following is a *résumé* of the foregoing:—

| | |
|---|---|
| Earth Backing, Specific Gravity ... | 1·8 |
| Brickwork, Specific Gravity... | 1·7 to 1·8 in lime mortar |
| Ordinary Stone Masonry ... ... | 2·1 |
| Special... ... ... ... ... | 2·25 |
| Granite ... ... ... ... | 2·5 to 3 in cement |
| $\phi$ ... ... ... ... ... | 1½ to 1 |
| $\phi_1$ ... ... ... ... ... | 3 to 1 |

**(16)** To revert to Fig. 6. $OB$ is the plane of repose set out at 1½ to 1 from the base line $OO_1$. The terrain line $AX$ being horizontal the plane of rupture bisects the angle $AOB$, and $\theta = \frac{AOB}{2}$. The prism of earth $AOX$ which exerts active pressure on the wall will do so by sliding in its base $OX$. The resultant line of pressure of this weight will act through the centre of gravity of the triangle $AOX$, and a line, shown dotted on figure, drawn parallel to the base of the triangle $AOX$ through $G$ cuts the back of the wall $AO$ at the point marked $a$. $Oa$ is clearly one-third of $AO$, and thus we see that whenever the apex of the triangle of earth pressure is coincident with the inner point of crest of wall, the resultant earth pressure invariably acts at one-third the vertical height of wall above base or at $\frac{H}{3}$. In Fig. 5 the apex of the corresponding triangle is not at $A$ but at $A_1$ higher up, consequently the centre of pressure will be at a greater height above the base.

**(17)** Having found $a$, the point of application of $r$, its direction is found by drawing $ab$ at right angles to $AO$ and from it setting off $r$ upwards at an inclination of $\phi_1$, or 3 to 1 from the normal $ab$. The line $r$ should be continued through the profile. This force will now have to be combined with that consisting of the weight of the masonry acting through its centre of gravity.

By far the easiest method of finding the centre of gravity of a trapezoidal figure is as follows:—From $A$ measure off $Ac$ = base $OC$ and similarly from $C$, $Cd$ = crest $AD$, join $cd$ and also the points of bisection of $AD$ and $OC$. The intersection of these lines is $G$, the centre of gravity of the profile $ADCO$. From this point $G$ draw a vertical, cutting $r$ at the point $f$.

The directions of the two forces $r$ and $W$ are now connected on the profile, and the next step is to find out their values.

Let $H$ denote the vertical height of the wall. Then the weight of the prism $AOX$ taken as one unit wide is $\frac{H}{2} \times AX \times w$, $w$ being the weight of a cubic foot of the earth backing. Similarly that of the wall will be $H \times W \times w_1$, $W$ being the half width of the wall. Let $\epsilon$ be the specific gravity of earth, $\rho$ that of masonry, then the weight of a cubic foot of water being 62·31 lbs. $w = \epsilon \times 62{\cdot}3$ and $w_1 = \rho \times 62{\cdot}3$. Hence the two expressions which have to be graphically equated become

$$H \times \frac{AX}{2} \times \epsilon \times 62{\cdot}3$$

$$H \times W \times \rho \times 62{\cdot}3$$

eliminating the common factors we have as representative of the loads on either side

$$\text{Earth} - \frac{AX}{2}$$

$$\text{Masonry} - W\rho$$

Reducing the earth expression to the base of the specific gravity of masonry, *i.e.* by dividing both expressions by $\rho$, we have

$$\text{Vertical earth pressure} = \frac{AX}{2} \times \frac{\epsilon}{\rho}$$

$$\text{Vertical wall pressure} = W.$$

**(18)** This arrangement for the simplification of expressions is commonly adopted in graphical calculations.

The force polygon is shown in Fig. 6a. Here the vertical force is represented by $ab = \frac{Ax}{2} \times \frac{\epsilon}{\rho}$. To effect this fractional multiplication graphically, $ac$ is drawn horizontally $= \frac{AX}{2}$. On $ab$ mark off on any scale, 1·8 the specific gravity of earth and on $ac$, 2·1, that of masonry, join these points and through $c$ draw $cb$ parallel to the last line. Then by similarity of triangles, $ab$ or $p$ will $= \frac{ac \times 1{\cdot}8}{2{\cdot}1} = \frac{AX}{2} \times \frac{\epsilon}{\rho}$. From $b$ set off $bd$ inclined at the angle $\theta$ from $p$, and through $a$ draw $ad$ parallel to the line $r$ in the profile Fig. 6. The triangle $abd$ will then represent the forces $p$ $q$ and $r$. Having now obtained $r$ in quantity as well as direction, it has to be combined with the force $W$. This can be best effected on the same diagram. The procedure is as follows:—Draw $de$ vertically $= W$, *i.e.* $= 7$ ft. and join $ae$. The triangle $ade$ will then be the force triangle required, $ad$ being $r$ and $de$ $W$; the resultant of these two forces $R$, will be the third side of the triangle $ade$, in direction and magnitude. $R$ is then the resultant line of pressure on base which is required. In Fig. 6 from the point $f$ draw $R$ parallel to $ae$ in Fig. 6a, cutting the base at the point $h$.

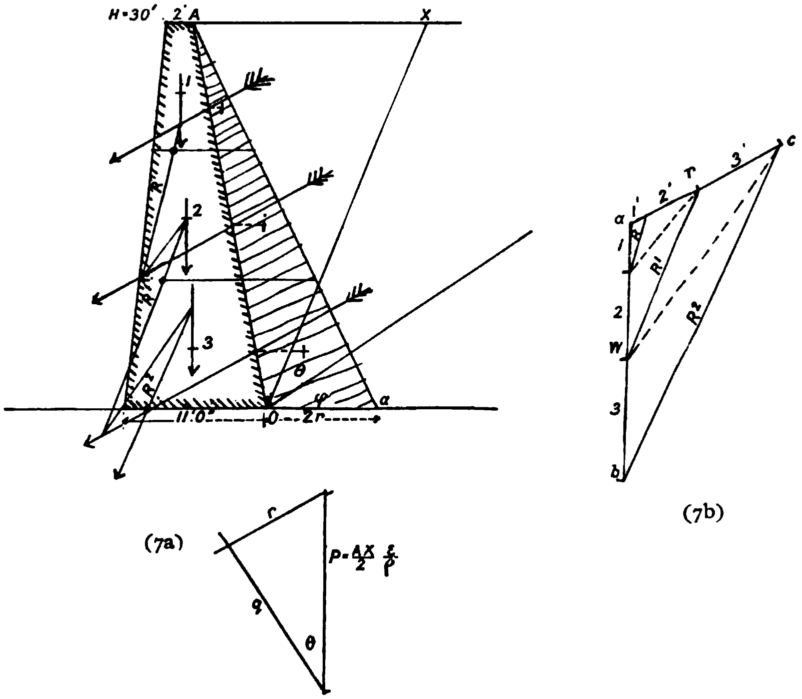
**(19)** The distance of $h$ from the outer toe $C$ will be found to be close by 1 foot 6 inches or $\frac{1}{8}$ base $CO$. When the wall is of brickwork of the specific gravity 1·8, the same as that of the earth backing, the force $p$ is drawn equal to $\frac{Ax}{2}$. This is shown in Fig. 6b. The resultant $R_1$ reciprocal to that in Fig. 6b is drawn as a dotted line on the profile Fig. 6. Its incidence on the base $CO$ is nearer the toe $C$ than that of $R$. To bring the incidence of $R_1$ back to the point $h$, which latter is at $\frac{b}{8}$ distance from $C$, the base width will have to be increased by half a foot and be in terms of $H$; $\cdot 4H + \frac{1}{2}$, instead of $\cdot 4H$, as it stands in the drawing.

### *The Reciprocal Triangle of Earth Pressure.*

**(20)** Hitherto we have only been concerned with the point of intersection of all the forces engaged on the base line of the retaining wall, the position of which point demonstrates the stability of the section, and which is identical in result with the analytical system of taking moments about the outer toe, or some other fixed point in the base line.

This procedure is sufficient in most cases, as it can be safely assumed that if the wall at the weakest part, viz., the base, is in stable equilibrium, the upper part will be still more so. When, however, retaining walls are of exceptional height, or are subjected to abnormal conditions of pressure, or are of unusual section, it will become desirable that the line of pressure be traced right through the section of wall from crest to base, thus enabling the designer by inspection of the weak points to modify the profile as may be expedient.

**(21)** In order to be in a position to draw the line of pressure, the distribution of the earth pressure on the back of the wall has to be ascertained, the previous method only giving the point of application of the resultant on the base.



FIGS. 7, 7a, 7b.

To effect this the triangle of earth pressure reciprocal to that of the earth pressing on the wall has to be drawn, reduced to a masonry base, so that ordinates drawn parallel to the base of this triangle to any point on to the back of the wall will truly measure the intensity of pressure at such point.

In Fig. 7 the triangle *AOX* is the area of earth backing, whose weight presses on the wall. The pressure developed by this prism is *nil* at the crest *A*, and is a maximum at *O* the base, hence the form will be that of a triangle having its apex at the crest *A*. The area of this triangle should equal the total earth pressure, and each ordinate drawn parallel to its base will measure the intensity of unit pressure at that point. Now the total earth pressure in terms of unit weight of masonry is, as we have already seen, $rH$, $r$, as in Fig. 7a, being the resultant of the earth pressure divided by $H$ (the weight being unity), thus the area of the triangle must $= rH$, *i.e.* its mean width will be $r$ and its base will measure $2\ r$. In Fig. 7, if $Oa$ is made $= 2\ r$, then the triangle $AOa$ will be the reciprocal triangle of earth pressure, its area being $rH$. The base $Oa$ being horizontal, the measure of the intensity of unit pressure at any point on the back of the wall $AO$, will be the horizontal ordinate of the triangle $AOa$. The direction of the pressure, however, is oblique, being parallel to $Ob$.

(**22**) The procedure of drawing the line of pressure will be briefly described. The wall and triangle of pressure are divided into laminas of equal depth, in this case three. Their common depth $\frac{H}{3}$ can then be eliminated from the graphical calculation, the weights of the laminas can then be represented in the force diagram 7b by their respective mean widths.

Thus in Fig. 7b the load line $ab$ or $W$ is composed of the three half widths of the wall 1, 2 and 3. In the same way $ac$ or $r$ is composed of the three half widths $1_1$, $2_1$ and $3_1$ of the laminas of the triangle of pressure. The incidences of $r_1$, $r_2$ and $r_3$ on the back of the wall are at the intersection of lines drawn through the *CG's* of the laminas. From these points the directions of the forces are inclined parallel to $r$ in Figs. 7a and 7b. From the intersection of $1_1$ with 1 in Fig. 7, $R$ is drawn parallel to its reciprocal $R$ in 7b, the point where this cuts the end of the base of the lamina 1, is a point on the line of pressure. From the intersection of $R$ continued, with $2_1$, a line is drawn back parallel to its reciprocal in Fig. 7b the resultant of the three forces 1, $1_1$ and $2_1$. Again from its intersection with the force 2 the line $R_1$ is drawn to meet the inclined force $3_1$ parallel to its reciprocal $R_1$ in the force diagram, this intersects the base of lamina 2, giving a second point in the line of pressure and terminates as before, at its junction with the inclined force $3_1$. The procedure as above described with regard to the reverse line gives the starting point of the final resultant $R_2$ which intersects the base of the wall. This latter point is clearly identical with that obtained by the simpler procedure formerly given.

(**23**) We will now proceed to show the method of obtaining the incidence of the resultant line of pressure on the base of a trapezoidal wall in cases where the terrain line is not horizontal and also where it is surcharged with a weight above the level of the wall crest.

Fig. 8 represents a case with the terrain inclined upwards. First the point $X$ or the intersection of the line of rupture with the terrain is obtained by the means described in par. 11. The area of the prism of earth pressing on the rear of the wall $AOX$ cannot be represented as was done hitherto by $\frac{AX}{2} \times H$—

but if the line $XX_1$ be drawn parallel to the back of the wall till it intersects the horizontal line $AX_1$ and the point $X_1$ joined with $O$, then the triangle $AOX_1$ is evidently equal to $AOX$ and its area is equal to $AX_1 \times H$.

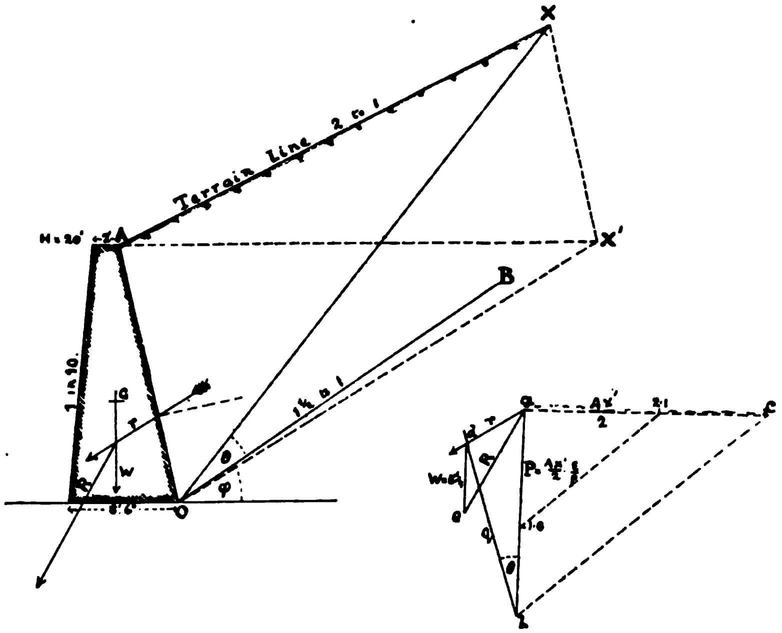


FIG. 8.

The centre of gravity of the prism is evidently on a line drawn parallel to the base $OX$, consequently the incidence of the inclined force $r$ on the back of the wall will be at one-third of $H$. The same is the case with any triangle having its apex at the point $A$. The procedure of finding $r$ is identical with that already described, $p$ being made equal to $\frac{AX_1}{2} \times \frac{\epsilon}{\rho}$, and $q$ set off at the angle $\theta$ from its extremity $b$, which intercepts the inclined line $r$ drawn from the origin $a$.

(24) In Fig. 9 we have a similar case, but with the terrain line inclined downward. The procedure is identical with that in the last case.

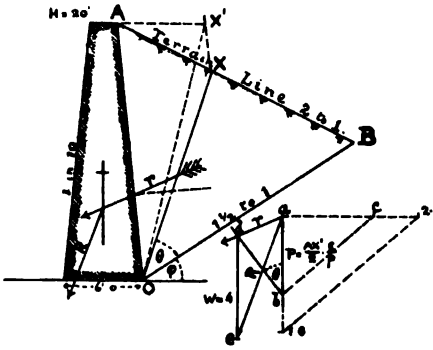
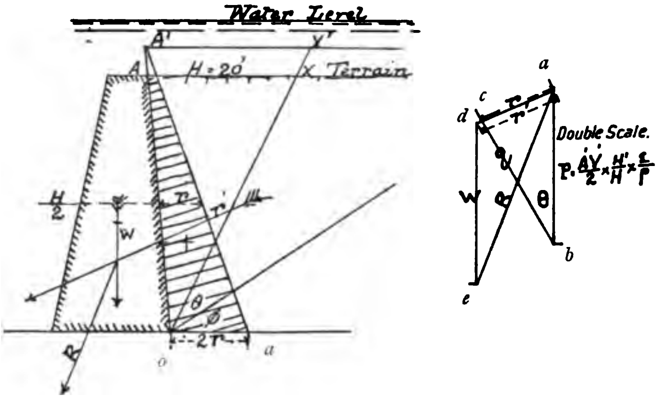


FIG. 9.

(25) Fig. 10 is a replica of Fig. 5, in which the terrain line is loaded with water. As already noticed in par. 13 the prism of earth acting on the wall is the triangle $A_1 OX_1$. Consequently in the force polygon Fig. 10a, $p$ is made either by graphical or arithmetical process equal to $\frac{A_1X_1}{2} \times \frac{\epsilon}{\rho} \times \frac{H_1}{H}$. The

last fraction enables $H$ to be eliminated as a common factor. By setting off $q$ at the angle $\theta$ from the base of the load-line the value of $r$ is obtained, as also the base $Oa$ of the triangle of pressure $AOa$, which is $2r$. The earth pressure is represented not by the triangle $A_1Oa$, but by the hatched trapezium $AOa$, consequently $r$, in Fig. 10 is not representative of the mean pressure, but $r_1$ the half-width of the trapezium which is greater than $r$ does so. Hence in Fig. 10a $r_1$ has to be measured off from the point of origin $a$, to $d$, and the vertical $W$ or $de$ is drawn through this point $d$, equal to the half-width of the trapezoidal wall; $ea$ is then the direction of $R$. In the profile



FIGS. 10, 10a.

Fig. 10 the inclined force $r_1$ cuts the back of the wall at the intersection of a horizontal through the centre of gravity of the trapezium $AOa$, not at that of the triangle $A_1Oa$.

**(26)** The consideration of further abnormal conditions to which retaining walls are subject will now be deferred until the principles of design are worked out in regard to trapezoidal walls of various heights and face batters under ordinary conditions of earth pressure. For this purpose Figs. 11 to 15 have been prepared, being a series of diagrams of sections of equal strength but of varying face batters, placed side by side for purposes of comparison. Inspection of these diagrams will at once demonstrate to the eye the influence exerted in the economical design of the sections by face batter, the diagrams also furnishing reliable types of sections for practical use, based on which formulas easy of application can be deduced.

In all cases the specific gravity of the material of the wall is taken as 2·1, that of the earth backing as 1·8, the top width at crest is a fixed dimension of 2 feet, and each section is so designed that the centre of pressure falls close to the outer toe at a distance of one-eighth of the base from that point. As noted in par. 19, if the wall is of brickwork with a specific gravity the same as the earth backing, the incidence of the resultant on the base will be beyond $\frac{b}{8}$ from the toe at a point about $\frac{b}{10}$ from the same point.

Thus brick walls built of these proportions will be safe though of less stability, or if the same statical condition as was deemed requisite in the case of stone walls be insisted on, the base should be increased from 6 inches in the higher to 4 inches in the lower sections, right through.

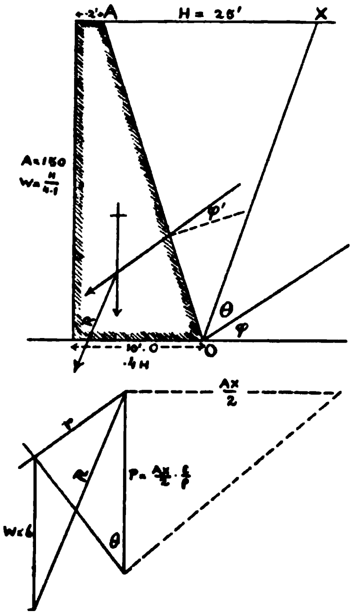


FIG. 11.

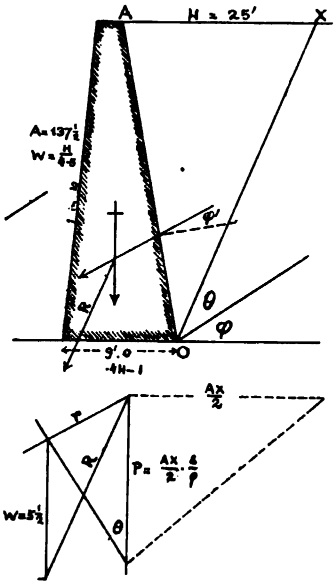


FIG. 12.

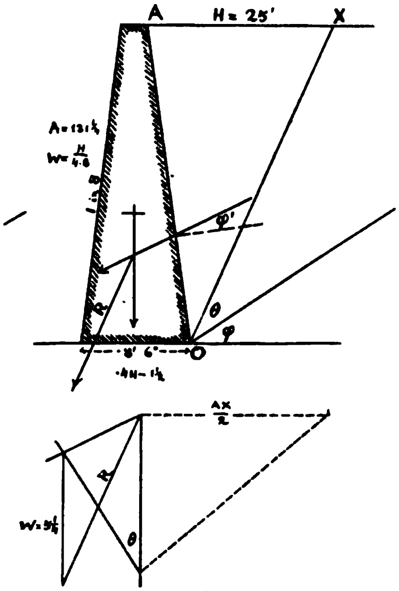


FIG. 13.

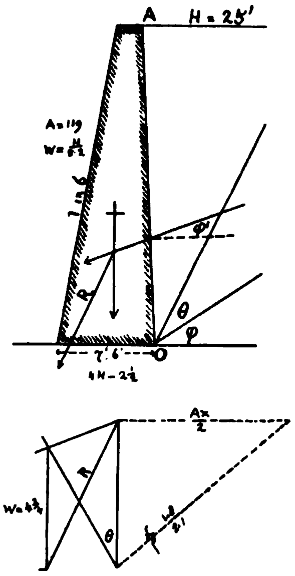


FIG. 14.

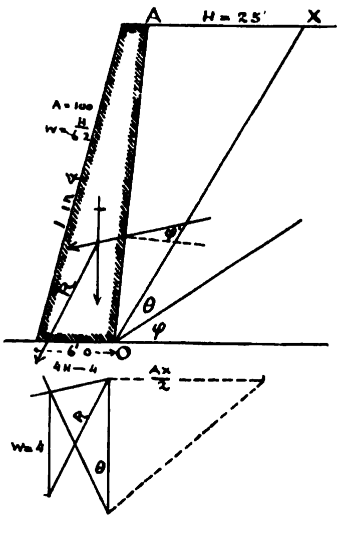


FIG. 15.

Reference to the diagram series will show that with the change of face batter the consequent gradual reduction of the base width causes the back slope to become steeper, till at last in Fig. 15 it vanishes and the back slope becomes inwards in sense. This diminishing of the base is so regular and so clearly governed by the height that the base width can well be expressed in terms of *H*, and rules can be framed giving the required base width of any wall as a fraction of its height. In this, the base for a vertically faced wall, as in Fig. 11, will be used as a standard, the whole of that series working out to ·4*H*. The base width, of Fig. 12 works out to ·4*H*—1, of Fig. 13 to ·4*H*—1½, of Fig. 14 to ·4*H*—2½, and of Fig. 15 to ·4*H*—4.

Sections have been drawn out for every depth from 30 to 10 feet for each face slope, though not given here, and the tabular statement below is the result.

TABLE I.

| Face. | *H*. | Base. | Area square feet. | $\frac{H}{W}$ | Height (*H*) when back becomes vertical. | Face. | *H*. | Base. | Area square feet. | $\frac{H}{W}$ | Height (*H*) when back becomes vertical. |
|---|---|---|---|---|---|---|---|---|---|---|---|
|  | Feet. |  |  |  | Feet. |  |  |  |  |  | Feet. |
| Vertical. | 30 | ·4*H* | 210 | $\frac{1}{4\cdot3}$ | 5 | Batter 1 in 10. | 30 | ·4*H*—1 | 195 | $\frac{1}{4\cdot6}$ | 10 |
|  | 25 | ·4*H* | 150 | $\frac{1}{4\cdot2}$ |  |  | 25 | ·4*H*—1 | 137½ | $\frac{1}{4\cdot5}$ |  |
|  | 20 | ·4*H* | 100 | $\frac{1}{4}$ |  |  | 20 | ·4*H*—1 | 90 | $\frac{1}{4\cdot4}$ |  |
|  | 15 | ·4*H* | 60 | $\frac{1}{3\cdot8}$ |  |  | 15 | ·4*H*—1 | 52½ | $\frac{1}{4\cdot3}$ |  |
|  | 10 | ·4*H* | 27½ | $\frac{1}{3\cdot4}$ |  |  | 10 | ·4*H*—1 | 25 | $\frac{1}{4}$ |  |
| Batter 1 in 8. | 30 | ·4*H*—1½ | 187½ | $\frac{1}{4\cdot8}$ | 12·8 | Batter 1 in 6. | 30 | ·4*H*—2½ | 172½ | $\frac{1}{5\cdot2}$ | 17·3 |
|  | 25 | ·4*H*—1½ | 131 | $\frac{1}{4\cdot8}$ |  |  | 25 | ·4*H*—2½ | 119 | $\frac{1}{5\cdot2}$ |  |
|  | 20 | ·4*H*—1½ | 85 | $\frac{1}{4\cdot7}$ |  |  | 20 | ·4*H*—2½ | 75 | $\frac{1}{5\cdot2}$ |  |
|  | 15 | ·4*H*—1½ | 49 | $\frac{1}{4\cdot6}$ |  |  | 15 | ·4*H*—2¼ | 43 | $\frac{1}{5\cdot4}$ |  |
|  | 10 | ·4*H*—1½ | 22½ | $\frac{1}{4\cdot4}$ |  |  | 10 | ·4*H*—2 | 20 | $\frac{1}{5}$ |  |
| Batter 1 in 4. | 30 | ·4*H*—4 | 150 | $\frac{1}{6}$ | Back is sloped inward throughout. |  |  |  |  |  |  |
|  | 425 | ·4*H*—4 | 100 | $\frac{1}{6\cdot2}$ |  |  |  |  |  |  |  |
|  | 20 | ·4*H*—4 | 60 | $\frac{1}{6\cdot7}$ |  |  |  |  |  |  |  |
|  | 15 | ·4*H*—3 | 38 | $\frac{1}{6}$ |  |  |  |  |  |  |  |

NOTE.—*H* = vertical height. *W* = mean width of walls (both in feet). $\rho = 2\cdot1$.

(**27**) In practice many revetment walls, as wings of bridges, have a crest sloping down at 1½ or 2 to 1 in uniformity with the side slope of an embankment which they support. If designed of equal strength throughout, in accordance with the base widths in the Table, the end portion of the wall in some cases would have an inward and the upper an outward back slope. Although it is possible to build the wall in this way, the outer slope gradually changing from inward slope to vertical and again outward, yet it would be a somewhat awkward construction, and for the sake of simplicity it would be as well to continue the wall as soon as the outer slope is merged into the vertical as one with a vertical back; this will cause the lower portion of the wall to have an excess of strength and material. This fact would militate against the use of excessive batter as 1 in 4, or even 1 in 6, for crest sloping walls, as the saving effected in the higher portion would be discounted by the excess in the lower. In the tabular statement given above the height at which the back slope disappears and the back becomes vertical is noted in the last column of the Table.

(**28**) The great economical advantage of face batter to retaining walls is clearly proved by the sections given and by the Table. For instance, the vertically faced wall in Fig. 11 has a sectional area of 150 square feet and a proportion of $\frac{W}{H}$ or $\frac{\text{mean width}}{\text{height}}$ of $\frac{1}{4 \cdot 2}$. Whereas in Fig. 15, of equal strength, the area is 100 square feet, and the fraction $\frac{W}{H}$ is $\frac{1}{6 \cdot 2}$. In this case the back of the wall has an inward slope, thus approximating to a pitched slope.

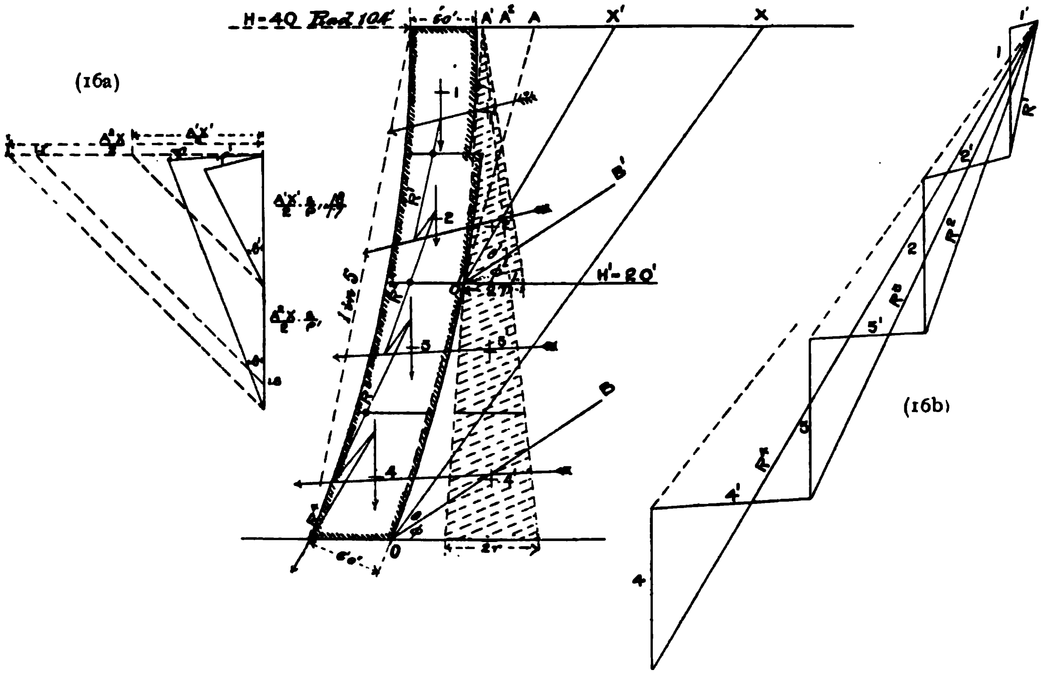
(**29**) Brunel's curved walls had a value of $\frac{W}{H}$ of nearly one-seventh, and leaned over to that extent, that the centre of gravity of the wall fell outside the heel of the base. A specimen of the section of one of these retaining walls is given in Fig. 16.

For the graphical calculation, the back of the wall is considered as consisting of two planes $A_1O_1$ and $O_1O$, $O_1$ being at half the height or 20 feet. above the base. As these portions of the back have different inclinations to the vertical, the directions of their several planes of rupture and values of $\theta$ and $r$ will also vary. Thus a separate graphical calculation will have to be made for each of the portions, viz., the whole $AO$ and the part $A_1O_1$.

The lower portion of the back $OO_1$ is continued up to meet the terrain line at $A$. The planes of repose $OB$ and $O_1B_1$ are then set out at 1½ to 1 with the horizontals from $O$ and $O_1$, and the planes of rupture $OX$ and $O_1X_1$ are found by bisecting the angles $AOB$ and $A_1O_1B_1$. The angles $\theta$ and $\theta'$, are obtained by this construction. The area of earth pressing on the whole wall is the four-sided figure $A_1O_1OX$, or the triangle $AOX$ + the triangle $AO_1A_1$. If the apex $A$ be moved to $A_2$ so that $AA_2 = AA_1 \times \frac{H_1}{H}$, then the enlarged triangle $A_2OX$, will equal the combined area of the two triangles just noted. $H_1$ being $= \frac{H}{2}$, the position of $A_2$ will be at the bisection of $AA_1$. Hence for purposes of measuring the area of the earth triangle, $A_2X$ will be considered as the base in lieu of $AX$.

With regard to the upper portion, the triangle $A_1O_1X_1$ represents the earth prism pressing on the upper half back $A_1O_1$, $A_1X_1$ will then measure the comparative weight of this part.

(**30**) To find the values of $r$ for each division the force triangle 16a is now constructed. The vertical load line is made $= \frac{A_2X}{2}$, for the whole depth of wall, and on the same line the value of $p$ for the upper half, viz., $\frac{A_1X_1}{2}$, is marked off. From the extremity of the load line and also from the higher point in it the angles $\theta$ and $\theta_1$ are set off, intercepting $r$ and $r_1$ on the two lines representing the directions of these forces, which lines are inclined at angle $\Phi_1$ above the normal to the planes $AO$ and $A_1O_1$ respectively.



FIGS. 16, 16a, 16b.

The next step is to construct the reciprocal triangles of earth pressure. With regard to the upper portion, $2r_1$ will form the base of the triangle, being drawn horizontally from the point $O_1$, the apex being at $A_1$ (*vide* par. 21).

The lower base could in similar manner be measured $= 2r$ from the point $O$, the apex of this larger triangle of pressure being at $A$. It will be more convenient, however, to make the inner side of this triangle coincide with that of the upper, viz., with the line $A_1O_1$ and $A_1O_1$ produced. The base $2r$ will then be measured from the intersection of $A_1O_1$ produced with the base line. It is clear that the area of the trapezium below $O_1$ will be the same, whatever inclination is given to its side, as all will have the same base and lie between the same parallels.

The procedure now closely follows that already described in the case of Fig. 7. First the wall is divided into parts of equal height, in this case 1, 2, 3 and 4. The triangle and trapezium of earth pressure forming the corresponding divisions of the wall and of the earth pressure are next found, and vertical lines drawn through the former and horizontal lines, *i.e.*, parallel to their bases through the latter. Through the points where these horizontal lines cut the back of the wall, the four feathered lines representing the directions of $r$ and $r^1$ are drawn.

(**31**) We are now in a position to construct the Haesslers force and ray polygon, which owing to the varied inclinations of $r$, presents some difference to that illustrated already in Fig. 7b. First, from the nucleus $O$ (Fig. 16b), the first inclined force $1_1$, is measured in its proper direction and equal in length to the half width of the triangle $1_1$ on Fig. 16. From its extremity, the vertical force 1 is set out equal in length to the half width of division 1 of the wall section. In the same manner the other forces $2_1$, 2, $3_1$, 3 and $4_1$, 4 are set out. The resultants, $R_1$, $R_2$, $R_3$ and $R_4$, are clearly combinations of $1_1$ 1, $1_1$ 1 $2_1$ 2 and so on, and the dotted rays drawn from the outer corners are the resultants of the odd numbers $1_1$ 1 $2_1$, $1_1$ 1, $2_1$ 2 $3_1$, etc. The drawing of the reciprocals on the wall section is identical with that already described with reference to Fig. 7, and need not be repeated. The intersections of $R_1$, $R_2$, $R_3$ and $R_4$ with the base lines of the divisions 1, 2, 3 and 4 give four points on the line of pressure. The relative position of these proves the stability of the wall. The weakest point is at the base, and it is evident that the stability of the wall would be considerably improved by the addition of an off-set from the face near the lower end.

(**32**) The centre of gravity of the whole wall falls without the inner toe. It is not shown in Fig. 16. Thus without the earth backing the wall would fall backwards. This was provided for during construction by tightly packing the back of the wall with rammed earth as the work proceeded. Several walls of similar light pattern were constructed by Brunel on the Great Western Railway.

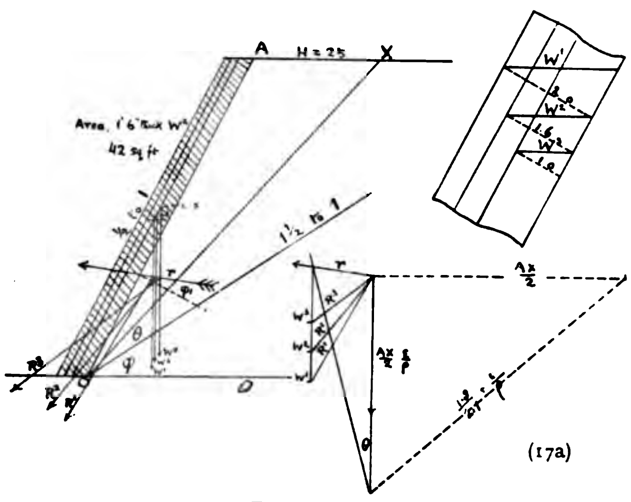
A curved face batter is very suitable for walls at or over 30 feet in height, as it corresponds more closely with the line of pressure and results in a more economical distribution of material. For irrigation works, however, this section is not very suitable except in the case of lock walls, which really belong to navigation canals.

(**33**) Retaining walls that overfall to such an extent as to be supported entirely by the earth, instead of *vice versâ*, are termed pitched slopes. Figs. 17, 18 and 19 are examples in question.

In Fig. 17 the pitched slope is ½ to 1, and the diagram exhibits tentative methods of ascertaining what thickness of pitching will be required. The procedure is identical with those already described for obtaining the centre of pressure on the base of retaining walls.

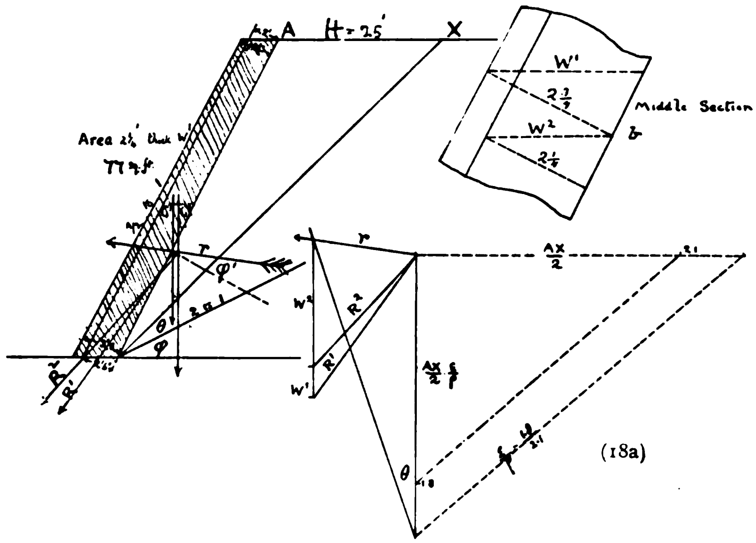
Trial is made of three thicknesses: 2 feet, 1½ feet and 1 foot.

*W* in the force polygon Fig. 17a has therefore three different values and consequently *R* has likewise three different directions and values. Of the three, $R_2$ intersects the base, consequently the intermediate thickness of 1½ feet is sufficient for purposes of stability.



FIGS. 17, 17a.

In Fig. 18 the slope is likewise ½ to 1, but φ, the angle of repose, is given an inclination of 2 to 1. In this example, two trial mean thicknesses of 2¾ and 2¼ feet are adopted, the pitching being made 6 inches thicker at the base than at the surface. Of the two thicknesses the greater, or $R_1$, will be the correct one to adopt.



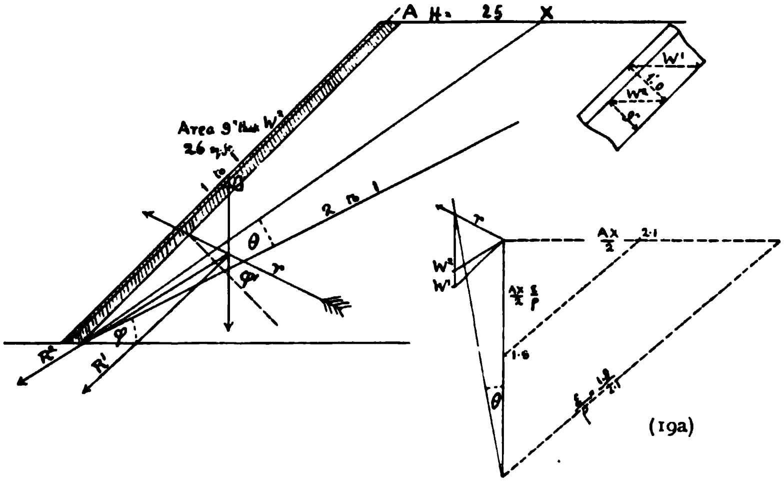
FIGS. 18. 18a.

In Fig. 19 the slope is 1 to 1 with the angle of repose 2 to 1, showing a 9 inch thickness of pitching is more than sufficient.

These examples prove the economy in the substitution of a masonry pitched slope for a retaining wall. A canal or escape fall designed on this principle is illustrated in Chap. IX.

(34) Varieties of retaining walls used in irrigation works may be classified as below as regards section :—

A, walls with horizontal crests with earth pressure to base.
B, walls with sloping crests with earth pressure to base.
C, walls with earth pressure for a fraction of height above foundations.
D, walls in parallel pairs termed land wings or straight returns.
E, single land wings or straight return stop walls with earth pressure equal on either side.
F, surcharged walls.
G, walls with unequal earth pressure on both sides.
H, weir walls with earth backing to near crest which is overlaid by water.



FIGS. 19, 19a.

Of these classes C, E and H are peculiar to irrigation works, not being found component parts of ordinary railway works. As regards class C this peculiarity is due either to a fall in the canal bed, at the canal cross work, when, as is generally the case, the foundations of the up-stream walls are taken down to the same level as that of the down-stream walls; the portion below the upper bed being thus built in a trench in the solid ground is not subject to earth pressure below that level. Walls of class E are used in falls, either river or canal, where there is no regulating or traffic bridge and the weir wall modified in section and raised to height of bank is run straight on into the embankment on either side. Class H has already been investigated in part (*vide* Fig. 10).

(35) Figs. 20 to 24 contain outline plans and elevations of various dispositions of walls in cross canal works.

Fig. 20 represents the skeleton outline of a level floor opening, either for a bridge to pass traffic or a regulating bridge with grooved piers and gates.

The archway, piers, etc., are not shown, as the object of the sketch is merely to illustrate the different systems of retaining walls, and the arrangement of the earth slopes. Figs. 20a are half plan and elevation of straight return or land wings similar to those used in railway over-bridges. These run back at right angles to the axis of the work at either side of the abutment, forming with the latter what is sometimes termed a U abutment.

This direct return type has this in its disfavour: that the crest being horizontal, the walls are of the same heavy section right through and, as ordinarily built, are not so economical as the sloping crested flank wall illustrated in Figs. 20b to 23b. In addition, in this design the flank walls form a guide for the water, a function which is valuable in case of increased current, as occurs below a fall.

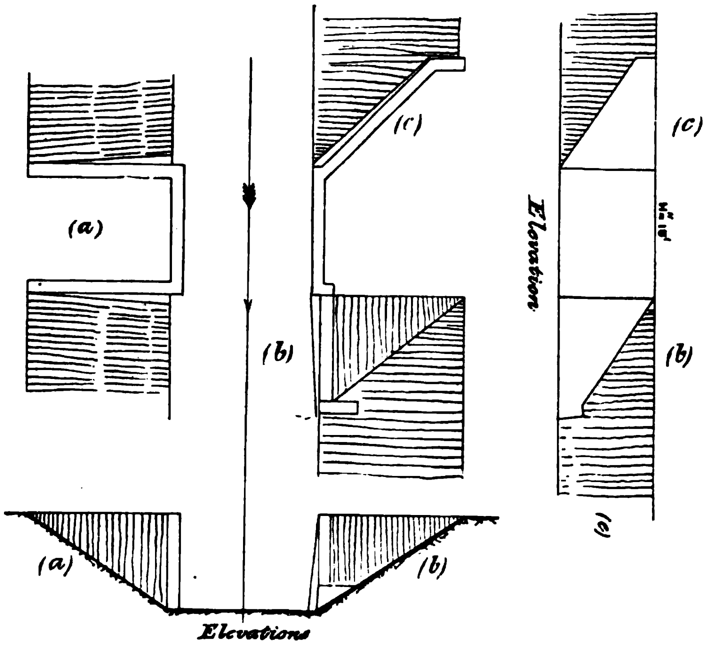


FIG. 20.

**(36)** Figs. 20c to 23c represent a new type of splayed wing, comparatively lately introduced, and it is now in much favour for various reasons for use up stream, generally in combination with (*b*) down stream.

This wall is naturally more expensive than a splayed and crest-sloped wall, but it has the advantage of great flank protection against soakage or percolation under a head of water, and in addition, by causing the widening of the embankment, enables the abutment to be made much shorter, or if the latter is already wide, carrying arches of a bridge, it forms a level space close to the work and clear of the roadway, which space is found useful for many purposes. It further guides the current into the opening, which is always less than the water surface of the channel.

With regard to (*b*) in plan and elevation in Fig. 20, these are views of a sloping wing with crest parallel to the axis of the work, the base or spring line of the batter being splayed outward at an inclination to the crest, of ratio of batter × ratio of fall of crest. In this sketch, the face batter being 1 in 10 and slope of crest 1½ to 1, the divergence of the base line from axis will be 1 in 15. The wing is provided with an end return 4 feet high, its length should equal its height × ratio of slope of bank, *i.e.*, 4 × 1½ = 6 feet. This return wall is surcharged, but with the face batter retained, at its low height it will be strong enough in section if built at the same thickness as the end of wing abutting on to it.

The advantage of a water wing of this description is obvious: it guides the water issuing through the opening in a straight course down the channel and also protects the banks down stream, from contact with water, except possibly at its lower extremity, which is removed far from the main work.

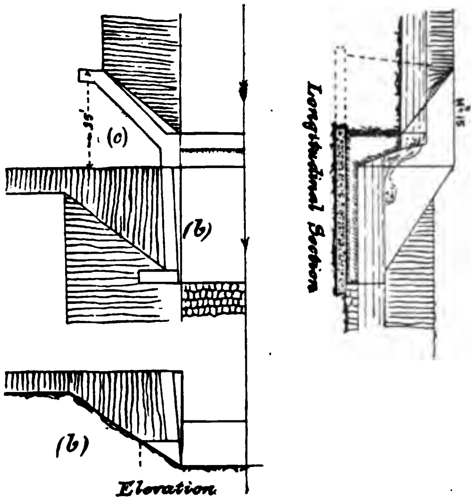


FIG. 21.

Where, as is often the case, a masonry floor extends down stream beyond the abutment, flank wings of this description are a necessity, or else with pitched slope a dwarf wing can be substituted.

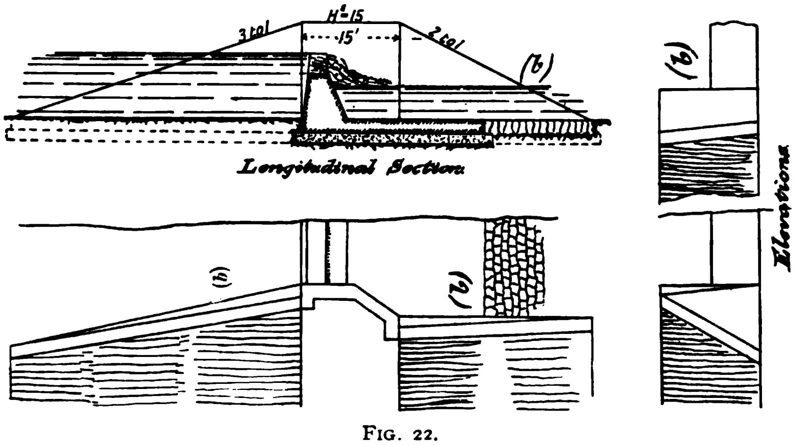


FIG. 22.

(**37**) In Fig. 21 a similar disposition of walls is exhibited as the *b* and *c* series in the last figure. In this case the floor is not level, but a drop of 4½ feet occurs in the canal bed. It will be seen that the abutments are very narrow, only sufficiently wide to cover the end of the cross drop wall. Here a direct return could not be adopted, or if so the abutment would have to be lengthened to contain the proper top width of bank approach. The efficient protection of the work from outflanking and the provision of the necessary width of bank are clearly effected in a satisfactory and economical manner by the splayed return (*c*).

Down stream the sloping crested water wings (*b*) display a slight difference in the way the batter is arranged. The base lines are parallel with the axis of the work, consequently the crest lines diverge inwards. This looks somewhat awkward on elevation, but there are cases where even a small outward splay of the water wings would be deemed objectionable, it being essential that the current be guided in an absolutely axial direction.

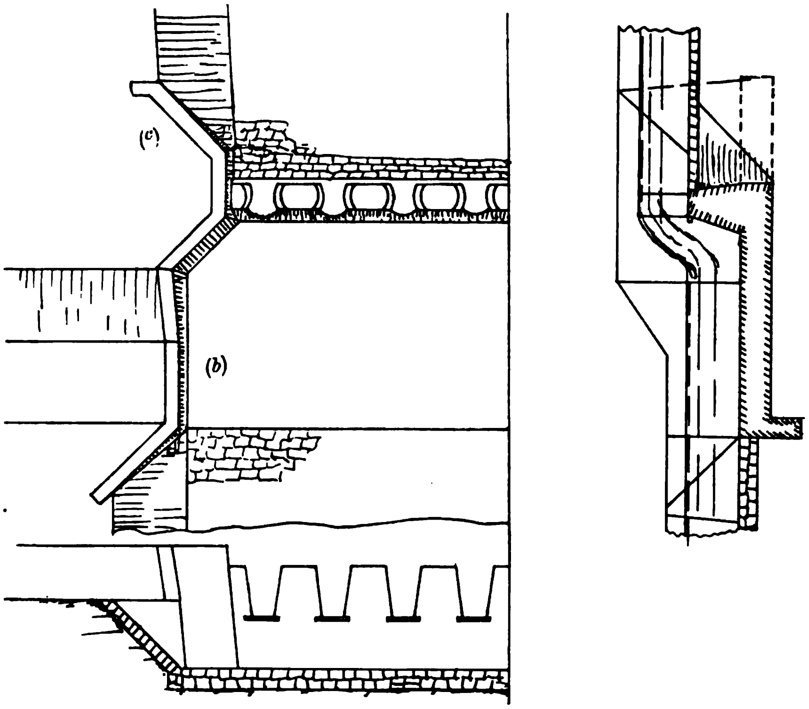


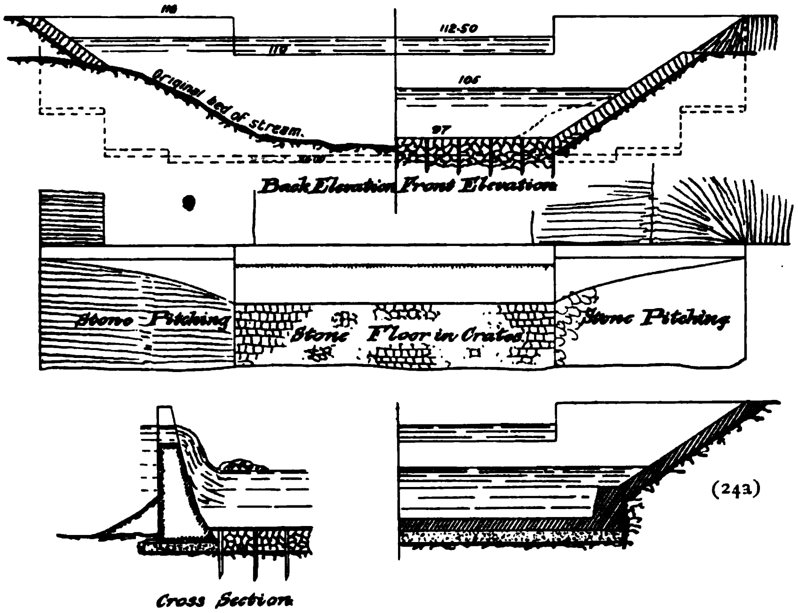
FIG. 23.

(**38**) Fig. 22 illustrates the case of a tank fall built in a depression which has the surface of ground at about floor level. In such cases the splayed returns up stream are not suitable, being necessarily of heavy section, and the connection with the tank embankment, which in this case has a flat inner slope of 3 to 1, and runs at right angles to the axis of the work, is rather awkward. An abutment of width equal to that of the embankment, combined with splayed sloping wings as illustrated, is about the best arrangement. In this case the down-stream wings (*b*) are shown set back, the outlining of the abutment being modified by having the corner bevelled off to prevent sharp corners and consequent eddies in the current. The dog-legged pattern down-stream wings shown in the next figure is a still better type.

The water wings (*b*) are both shown without returns, running right down to the toe of the embankment on either side.

They are sometimes constructed splayed as far as end of down-stream floor, and thence continued parallel to axis, or else curved as railway under-bridge wings, the latter arrangement coinciding with the old Madras practice. The curved walls are, however, rightly tabooed in modern works as being conducive to pooling, *i.e.*, a destructive rotary movement in the water which has to be guarded against.

(**39**) Fig. 23 is a modification of the conditions prevailing in the last example, showing the level of natural ground as 5 feet higher, the down-stream channel being in cutting. In cases such as this the splayed returns (*c*) can be employed up stream with advantage, the length of abutment being only just sufficient to contain the end of the weir wall and the splayed returns arranged so as to afford the necessary width to the embankment.



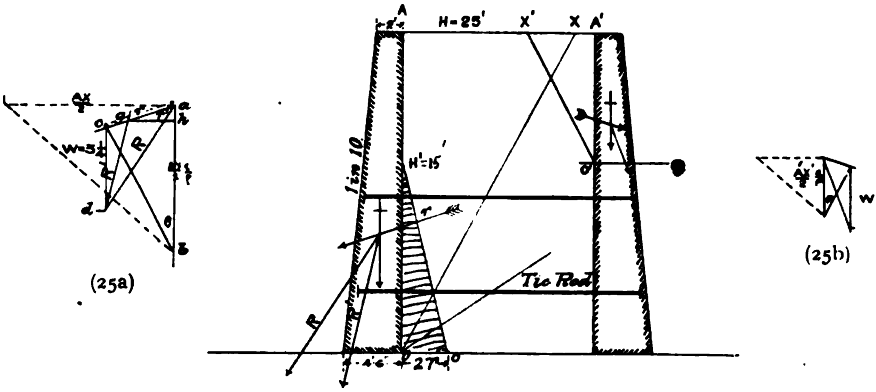
FIGS. 24, 24a.

The down-stream wings are of the dog-legged pattern, the abutments being also battered as well as the wings. This arrangement, which has been adopted in the designs for canal falls given in Chap. IX., is probably the best for this purpose.

(**40**) Fig. 24 contains elevation plan and section of a weir across a river which was constructed by the author in Burma. This illustrates class E. There being no wing walls proper, the throw back of end of embankment is effected by continuing the weir in either direction with a raised crest, the single flank walls thus answering the purpose of the double land wings. In this case the flank continuations were stepped up in foundation, and owing to the water pressure to which they are subject, were carried on of the same section as the weir proper.

The banks below this weir were pitched with stone, and the floor was formed of cribbed stone planked over and further protected by a subsidiary crib weir of loose stone, so as to form a water cushion. This style of construction was necessitated by circumstances, and can hardly be deemed otherwise than a temporary arrangement, although it has lasted some years. The half section marked 24a shows the probable eventual substitution of a masonry floor flanked by dwarf walls and stone pitching laid in mortar. A similar design for a canal fall is exhibited in Fig. 4, Chap. IX.

Having thus touched upon the subject of the disposition of retaining walls as are commonly used on canal works on plan and elevation, it is now proposed to continue the investigations of walls under abnormal conditions of earth pressure.



(25a) (25b)

FIGS. 25, 25a, 25b.

(41) Fig. 25 represents the section of a pair of land or direct return wings, such as are commonly employed in railway over-bridges. It will be seen that *OX*, the plane of rupture, clears the opposite wall, and consequently the earth pressure on *AO* is identical with that which would take place were the opposite wall non-existent. This would apply equally to either wall, and so, notwithstanding the reduced width of the earth retained, no diminution of the section of the walls is admissible. This fact causes this type of wall, if of any considerable height, to become an expensive construction, and so it is superseded where practicable, as already noted, by sloping wing walls. Owing, however, to the small space between the return wings, which seldom exceeds 20 feet, all undue earth pressure can be neutralised by the simple device of connecting the walls together with iron tie-rods. If this is done, a very light section of wall only need be provided, and thus equipped, the land wings can compete successfully as regards economy of section with the other types.

**(42)** In Fig. 25 the walls are shown of reduced section, the face batter being 1 in 10, with back vertical. In Fig. 25a the usual triangles of forces *abc* and *cda* are constructed, giving the values of $r$ and $R$. The reciprocal of the latter resultant transferred to the section falls well beyond the outer toe of base. In designing the section of the tie-rods required, credit should be given to the wall for the proportion of the horizontal component of $R$ absorbed by it, the balance being provided for by the tension in the tie-rods. A simple graphical process will solve the point. According to the rules adopted for the design of retaining walls, the resultant $R$ should cut the base ⅛ of its width recessed within the toe. If this proportion be set off on the section, and a line $R_1$ drawn from this point to the intersection of the feathered arrow $r$ with the vertical through the centre of gravity of the section, $R_1$ will then represent the proper inclination of the modified resultant. Now on Fig. 25a draw a line *dg* parallel to $R_1$ from the extremity of $W$, intersecting $r$ at the point $g$. The intercept *cg* then represents the proportion of $r$ which is absorbed by the wall and the remaining *ga*, that which has to be neutralised by the tension of the tie-rods. As, however, the tension is horizontal, the horizontal equivalent of this inclined force needs only be considered; *gh* or $r_1$, drawn horizontally, will then represent this force.

Reverting to Fig. 25, with a value of $H_1 = 15$ feet, the line of pressure will fall at about the required distance within the toe of that base, so that above this point there is no necessity for extraneous assistance. Below, however, the line of pressure will trend towards the face, eventually falling outside altogether, as is shown by a final resultant $R$. The reciprocal triangle of horizontal earth pressure, reduced to the corresponding area of masonry, will be a triangle $H_1OO$, having its apex at $H_1$, 10 feet below crest, and the width of its base being marked off $= 2r_1$ on Fig. 25a.

**(43)** The total horizontal stress to be carried by the ties per unit length will be the area of the triangle $H_1OO \times 62{\cdot}3 \times \rho$, but as it is proposed to place the ties 5 feet apart, the stress to be provided for will be the above expression multiplied by 5, or $\frac{3{\cdot}8}{2} \times 15 \times 2{\cdot}1 \times 0{\cdot}28 \times 5$ tons $= 8{\cdot}4$ tons. The quantity ·028 is the weight of a cubic foot of water expressed in fractions of a ton, as 62·3 is the weight in lbs.

Two tie-rods will be required; the lower one will be placed at the centre of gravity of the pressure triangle, *i.e.*, 5 feet above the base of the walls. This tie will neutralise the whole of the horizontal stress, viz., 8·4 tons, but another will be required at a higher level to counteract the overturning movement above the 5 feet level, as the wall might break off here, and fail by revolving on its outer toe. The area of that portion of the triangle lying above the lower tie level is one-half of the whole; of this one-third will be transmitted to the apex, two-thirds to the base. Hence the upper tie will carry one-sixth of the whole, *i.e.*, 1·4 tons, and the lower the remaining five-sixths, or 7 tons.

Taking 5 tons as the safe tensile strength of iron, the upper tie rod should have a sectional area of ·3 square inches and the lower 1·4 square inches. One ⅝ inch diameter rod would answer for the upper and 1¾ inch for the lower, placed at 5 feet intervals.

(**44**) The reduced area of land wings is 82½ square feet, as against 137½ of the normal section (*vide* Table I.), and $\frac{W}{H}$ is one-thirteenth only. This shows the great economy effected by the introduction of cross ties, the cost of which is trifling. The ties should be provided with large plate washers, and have a head at one end and screw nut at the other. The washers need not appear outside the wall, being placed in a recess near the face, which can be closed up when the final tightening has taken place.

The reduction in section rendered possible by this arrangement leads to the more extended use of this description of wall, particularly in the case of masonry pitched slopes being substituted for water wings, a style of construction used for river works, but which could be well adopted for canal falls or other cross-canal works.

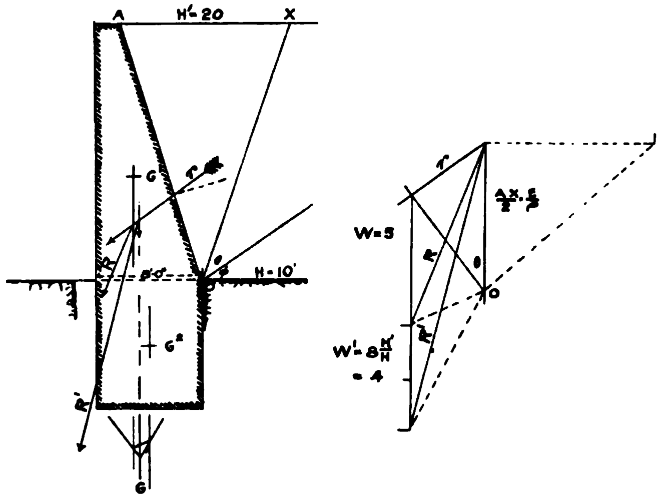


Fig. 26.

(**45**) In canal works in which a drop exists in the bed, it is a common practice to take the foundations of the up-stream wings down to the same level as those situated below the fall. The object of this arrangement is to obviate any possibility of saturation of the bank and percolation of water through the flank of the work. In Figs. 21 and 23 this is shown in the splayed up-stream returns (*c*), common to both plans. As canal falls are located so as to be in moderate cutting, the lower portion of the splayed walls is subjected to no earth pressure below the level of the bed of the upper reach.

These walls, then, belong to class C, *i.e.*, walls subjected to earth pressure for only a portion of their depth. As misapprehension undoubtedly exists regarding the action of forces in such cases, as is clear from many existing examples, it will be well to devote more than usual attention to the design of walls under similar conditions.

(**46**) Fig. 26 illustrates the typical case of a vertical wall 30 feet high, the lower 10 of which is sunk in a trench in the solid ground, and can consequently be safely considered as free from earth pressure below this level. The base width at $O$ is made $\cdot 4H$, in accordance with the tabular formula, and the foundations are carried down vertically. The resultant of the upper portion $R$, cuts its base at the proper point, but the final resultant $R_1$ falls without the base of the whole wall.

(**47**) Fig. 27 represents exactly similar conditions, but in this instance, the wall face being battered at 1 in 8, the base width is $\cdot 4H - 1\frac{1}{2}$, and below the level $H_1$ the back is carried down vertically, while the face batter continues. In this case $R_1$, the final resultant, cuts the base of the whole wall within the outer toe, proving the stability of the section, though it would be improved by still further widening the base, giving an increased batter below $H_1$, as is shown by the dotted line.

The area of the wall in Fig. 26 is 180 square feet, that in Fig. 27 is 156 square feet; the former is in unstable and the lesser section in stable equilibrium. If the base of Fig. 26 were widened to bring $R_1$ within the toe at the proper place, the discrepancy of areas would be still further intensified in favour of the lighter section. This proves the absolute necessity of having walls of class C designed with a face batter.

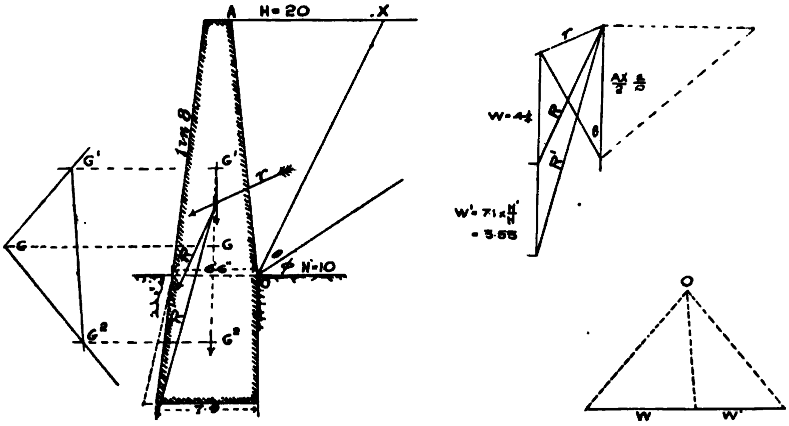


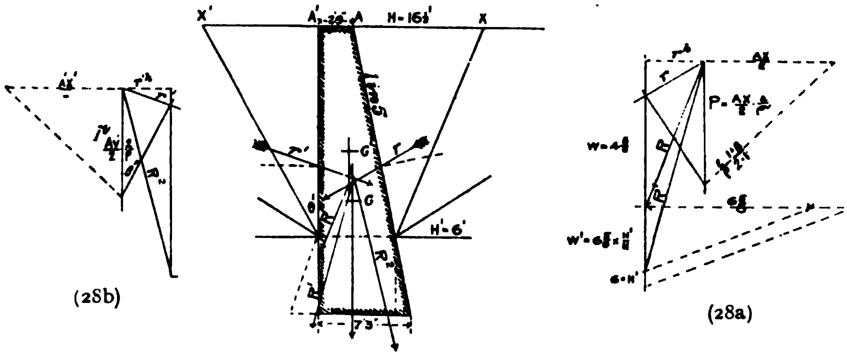
FIG. 27.

(**48**) As illustration of the too common neglect of this point, a few examples of actual sections will now be given and analysed.

Fig. 28 represents the section of a retaining wall on an existing canal fall with vertical face and sloping stepped back, which latter has been converted, for simplicity sake, into an equivalent batter. The wall is of brickwork, and has been credited in the graphical calculation with the usual specific gravity of 1·8, that of the earth being taken at the same value. It will be noticed that the final resultant $R_1$ falls just without the outer toe of the base. The wall should therefore have a further widening of the base in front and corresponding reduction behind. The front addition would have to be about double that lopped off the back, as the latter operation would tend to throw the centre of gravity forward, and this tendency would be further accentuated by the addition made in front.

**(49)** Now, supposing the wall reversed in section, *i.e.*, keeping to the same dimensions. Let the earth pressure be on the vertical side. The effect is duly worked out in Fig. 28b, and $R_2$ is the final resultant. This, it will be seen, cuts the base 2 feet within the outer toe. The great effect of the reversal of the section is plainly manifest. Not only is the shape of the section thus reversed more suitable to sustain the thrust of the earth, but the area of the triangle of earth pressing on the wall is also much reduced. The greater the back slope, the larger is also $AX$ and $r$. This is somewhat discounted by the greater inclination of $r$ to the horizontal when the back is inclined; this inclination is the graphical allowance for the weight of earth supported on the back slope of the wall.

With a vertical back, $p$ is less than with an inclined back, but the great difference in the two values of $r$ and $r^h$, the horizontal component, is due to the angle $\theta$, which is at a minimum with a vertical back.



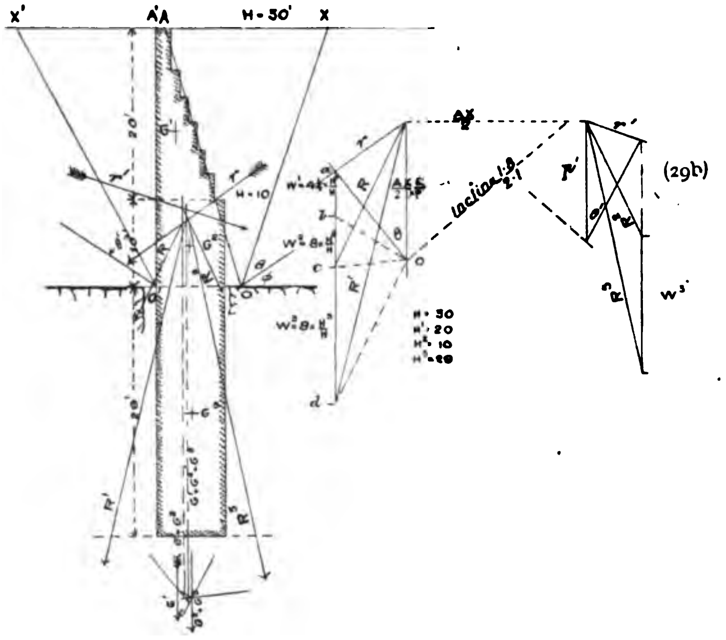
FIGS. 28, 28a, 28b.

**(50)** Thus we see that a vertically backed wall is the most economical, the batter being all thrown out on the front; it is only surpassed when the back, as in the case of Fig. 15, has an inward deflection. Exactly the same, as will be seen later, applies to dams subject to water pressure, the vertical or nearly vertical backed wall being the most economical design.

In Fig. 28, with pressure acting on the right side of the wall, the section is too weak; whereas if the earth pressure were to act on the opposite side, the wall would show excess of strength and could be sensibly reduced in section. The base at $H_1$ need not exceed 4 feet, *i.e.*, $4H-2\frac{1}{2}$ (*vide* tabular statement). The area of the section would then be, with 2 feet top width, about $76\frac{1}{2}$ square feet; whereas in Fig. 28 it is $112\frac{1}{2}$ square feet, and, further, the stability would be greater. The section in question could not have been designed on any principle except that of making the base one-third of the height, which it scales almost exactly. It is just strong enough for a height of $H$, but not for $H+H_1$, although the earth pressure stops short of the lower base.

**(51)** Another actual example of defective design is exhibited in Fig. 29.

This section has a slope of earth in front of the lower 10 feet; this is the tail end of the fore slope of a canal spoil bank which runs down the wall in question. At the commencement of the wall this slope disappears. Thus for half the length of the wall this slope has no appreciable influence in favour of its stability, and so has been omitted from consideration. The designer, however, has evidently treated it as a solid bank with horizontal terrain and has started the verticality of the back of wall from this point instead of from 10 feet lower down, where the surface of solid ground is reached; consequently that portion of the wall under direct earth pressure has been designed for a value of $H$ of 20 feet, instead of 30 feet, and the 8 feet base is thus too narrow, $R_1$, the resultant line of earth pressure, as shown in Fig. 29, falling without the base. The value of $\frac{W}{H}$ is barely one-sixth, a ratio far too low for a vertically faced wall. At the base of the whole wall, which is 58 feet high, the final resultant $R_1$ falls over 5 feet outside the outer toe. The error here consists as usual in altogether ignoring the action of the earth pressure below the surface line.



FIGS. 29, 29a, 29b.

If the wall were reversed the resultant $R_8$ reciprocal to $R_8$ in Fig. 29b, would also fall outside the base, but not to so great an extent as before.

(52) An attempt will now be made to show how the wall might have been designed, of much the same sectional area, but irreproachable on the point of stability, and under exactly the same conditions as before.

The wall profile in Fig. 30 is the result of two or three trials. A continuous face batter of 1 in 7 is given to the front. The back above the point $O$ is made parallel to the face, and below that point it is vertical. The object of making the upper part a rectangle and not a trapezium is in order to keep the base thickness of very moderate dimensions, as otherwise, the back below the point *O* being vertical, the lower section would become unnecessarily heavy. The inward back batter also reduces the earth pressure. The double back slope could be avoided if the portion below the point *O*, *i.e.*, in the trench, were also built with an inward batter. A vertical back line for this part is, however, considered more suitable. The two resultants in this design fall well within the base in both cases. The area of the wall in Fig. 20 is 397 square feet, that of Fig. 30 is 412 square feet, a slight excess.

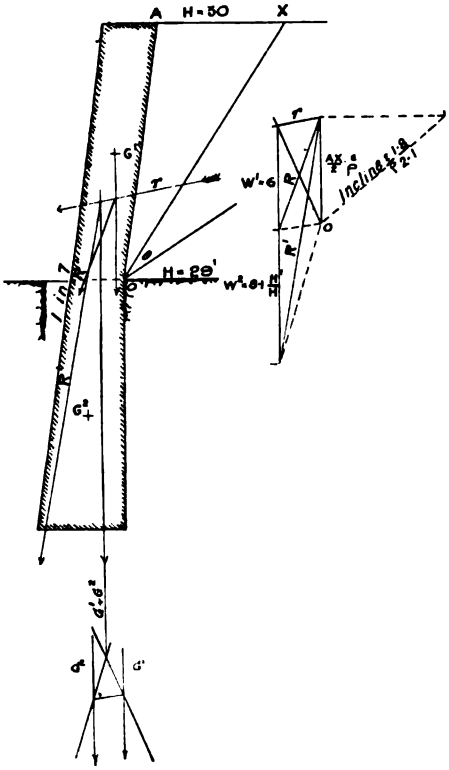


FIG. 30.

(**53**) As already observed, it is a common practice peculiar to irrigation works to take the foundations of the up-stream wings in case of a drop in canal bed, right down to the lower level; consequently walls above the drop wall or weir are not subjected to earth pressure below the level of the canal upper bed.

Fig. 31 represents a case in point and is a section of the splay up-stream wall (*c*) shown in Fig. 21. That figure represents a canal notch fall. The drop in the bed of canal is 6½ feet, the depth of water passing over weir, owing to the notches, is upheld to the normal depth, *i.e.*, 4 feet; another 4 feet is given as free board; the total height of wall above concrete then comes to 16 feet 9 inches, or say 17 feet. Of this the upper 8 feet is subjected to continuous earth pressure. Fig. 31 is a simple section designed to meet these conditions. Its strength is greater than necessity requires and could be economised if the top width were diminished, or an inward back slope given, neither of which are desirable.

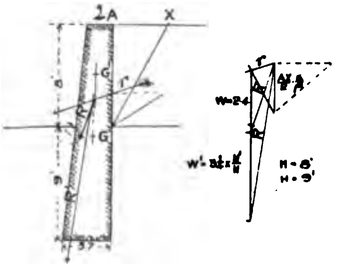
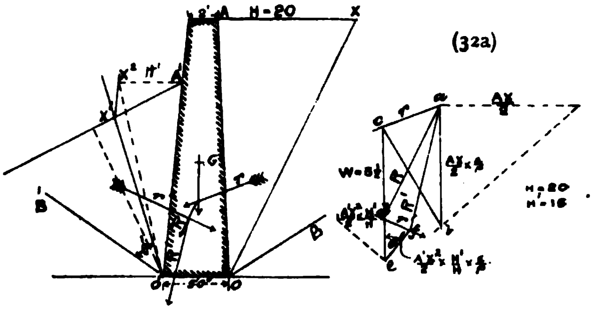


FIG. 31.

(**54**) One further problem is required to be solved before closing the subject of the sections of retaining walls, and that is the case where earth pressure exists on both sides of a wall acting in opposition, viz., class G. The method of combining these forces with the weight of the wall is illustrated in Fig. 32.

The profile represents a 20-foot wall with earth pressure up to crest at the back, while part of the face is covered by a falling slope. This case is identical with that prevailing in connection with land wings, at the ends of which the earth slope on the face reaches up to the outer crest and pivoting round on this point forms a half cone, pressing on the face of either land wing, thus forming in elevation a slope against the face.

In Fig. 32a, *a b c d* forms the usual combination of force triangles closing in $R$. We have now to combine $R$ with $r_1$, the inclined force acting in the opposite direction. This combination can be effected on the same figure by continuing the load line *c d* so that *d e* equals the relative area of earth weight which acts on the face of the wall. This will be $\frac{A_1X_2}{2} \times \frac{H_1}{H} \times \frac{\epsilon}{\rho}$. Then from the extremity *e*, the $\theta$ angle is set off, intersecting the inclined line $r_1$ drawn through *d*, at the point *f*. This latter joined with the starting point *a*, gives the direction and comparative value of $R_1$ the final resultant, *i.e.* the resultant of $r_1$ and $R$. In Fig. 32, $R$ is drawn as usual from the intersection of $r$ and $G$, till it meets the line $r_1$. This last point is the starting point for the final resultant $R_1$ which is drawn through the base Fig. 2. When the earth pressing on the wall has a horizontal terrain, as is the case in many direct return wings, the procedure is the same except that $O_1X_1$ bisects the angle $A_1O_1B_1$.



FIGS. 32, 32a.

## *Sloping Wings on Plan and Elevation.*

(55) The construction of trapezoidal wings with sloping crests in exact accordance with the proportions tabulated is a matter of no practical difficulty whatever. Figs. 33 to 36 contain sections and side elevations of a series of walls, the slope of crest being taken at 2 to 1. Reference to the sections of each height, at intervals of 5 feet, from 30 feet to 10 feet, which are joined in one figure (1a) clearly show that the back batter varies regularly, becoming slightly steeper with each decrease in height, till at a certain point it becomes vertical. The height of wall where the back batter ceases is given in Table I. For instance, with series A having a vertical face, the height is 5 feet. This, with a slope of crest of 2 to 1, will fall at a distance of 2 × 5, or 10 feet from the toe of slope, or if measured from the abutment the distance will always be 2 $(H\text{-}h)$, $H$ being the extreme height at beginning of wall. The position of $h$ is marked on each of the Figures 33 to 36 on elevation and plan.

(56) To align the winding back slope of the walls, all that is necessary is to mark the foundation outline, and erect one profile of the batter at the

abutment end, and another, vertical, at the point *h*. If the top ends of these batter profiles be joined by a taut cord representing the inner slope line of the crest, a straight edge applied to the cord above, and to the outline of base of batter will give the inclination at any point. The building of the back batter will be just as easy as that of the face. If the courses are inclined as should be the case, *i.e.* normal to the face, each single course or two courses can be set back in accordance with the profile, thus affording a slightly

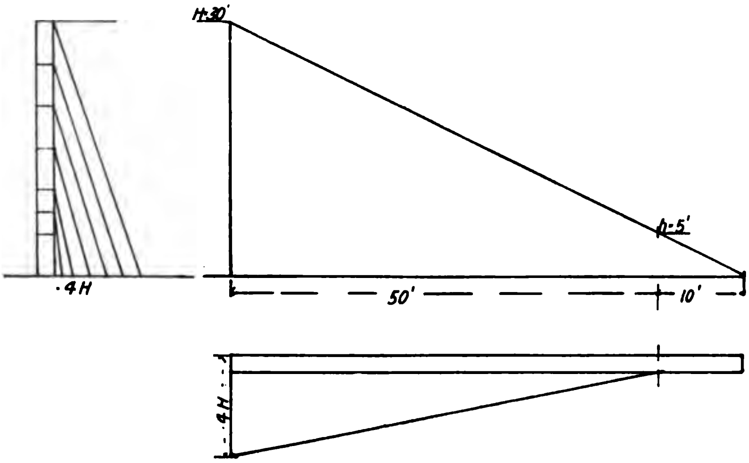


FIG. 33.

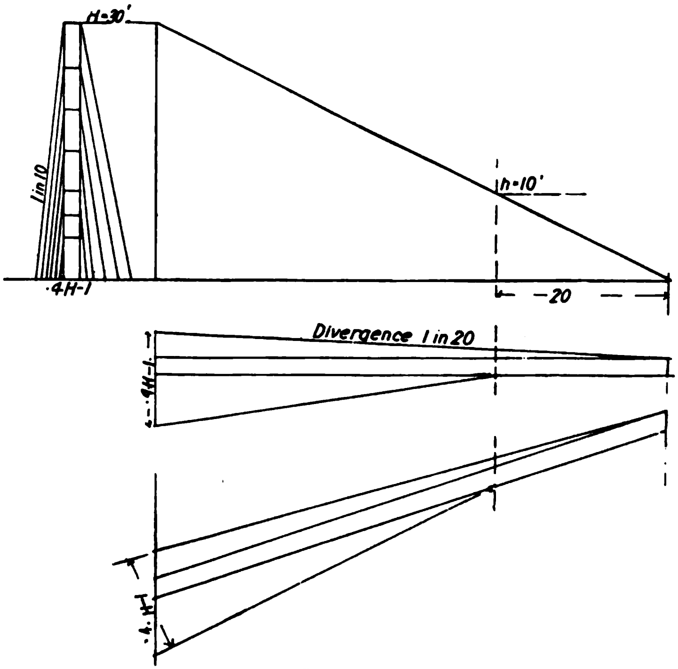


FIG. 34.

stepped or roughened face, which is of no disadvantage and is concealed from view. Walls built with inclined backs are in fact easier to construct than the stepped backs, which custom has prescribed for retaining walls, with their numerous insets and offsets, all requiring measurement, whereas in the former case no measurements of varying thicknesses are needed, the shape of the wall following the profiles automatically, as previously described.

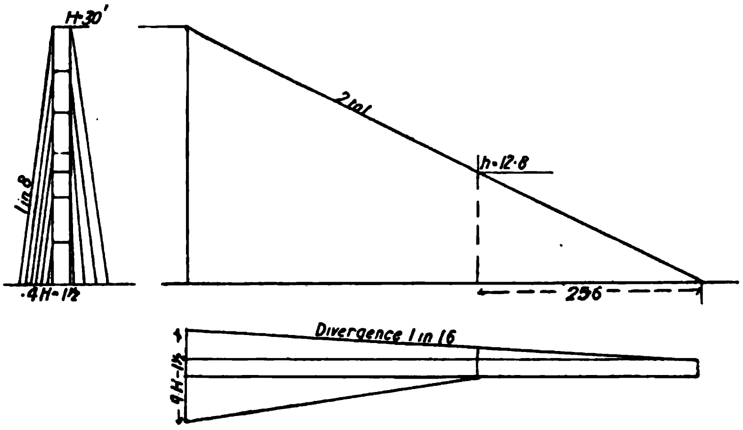


Fig. 35.

(57) The author is strongly in favour of the adoption of inclined backs for retaining walls instead of vertical stepped backs. In the former case the earth backing, on settlement, closes firmly against the inclined surface of the wall, whereas with stepped backs the settlement is necessarily uneven and hollow spaces are certain to be formed; this has been noticed in actual cases, deep holes having been found adjoining the wall, the bank in settling having evidently cut itself clear of the offsets of the back, leaving spaces down which the water finds its way, causing further settlement and disintegration.

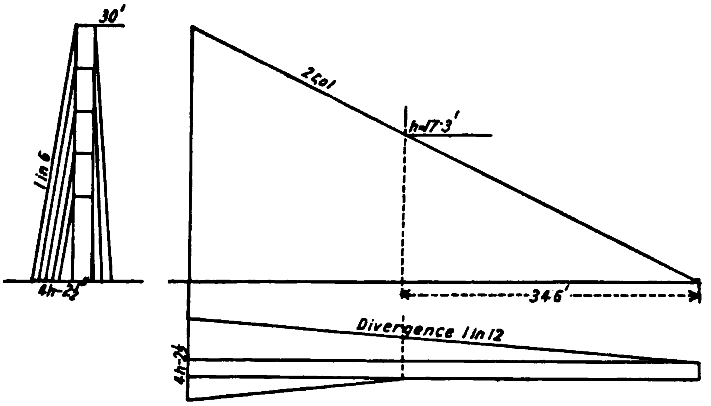


Fig. 36.

The advocates of stepped backs for retaining walls allege that the system possesses the following advantages: firstly, of increasing the friction of the earth against the wall, thus adding to its stability; and secondly, of affording a more solid support to the superincumbent earth backing, which can thus be credited to the wall as an increment to its weight. Both these assumptions are purely chimerical. The unequal settlement which is sure to occur with stepped backs to the wall must tend to decrease instead of increase the co-efficient of friction. As, instead of the whole of both surfaces being in close contact, they will be so only at intervals, the second consideration is not based on scientific grounds. The battered back, whether in steps or in one plain surface, performs identically the same statical function, viz., its resistance to external forces must be normal to its surface. Whether this surface is composed of several horizontal and vertical portions, or is in one plane, its reflex action is the same, as the vertical and horizontal forces if resolved, are identical in direction with the force of resistance normal to the plane surface. The weight of earth supported is duly credited to the wall by the normal inclination of the force $r$ to the back, which deflects the direction of this force downwards. With a wholly vertical back the direction of $r$ would be horizontal, if the friction of the two surfaces were neglected. In such cases no earth rests on any part of the wall. Thus it is clear that the stepped back possesses no advantages whatever over the plane batter, while it is undoubtedly more troublesome to design and to build.

# CHAPTER II.

## DAMS (SECTION).

### (I.) GRAVITY DAMS.

(1) WHERE water and not earth is upheld by retaining walls, its properties as a fluid and its weight being definitely known, there can exist no uncertainty on the subject of the actual pressure exerted.

A dam may be defined to be a wall of masonry or concrete which upholds a mass of water at its rear, while its face or lower side is free from the presence of water to any appreciable extent.

The waste water of the reservoir formed by the dam is disposed of in another direction by means of a waste weir or bye-wash, or in rare cases by means of sluices through the body of the dam.

The question of the most economical form of profile to resist the stresses induced by the pressure of the water combined with the weight of the wall itself has already received a good deal of attention from various scientific writers, and may be considered to be definitely established.

The pressure of water on the back of a wall at any point varies directly as the depth, so that the water pressure can be represented by an equilateral triangle with its base equal to its height, the base being normal to the rear face of the wall.

The unit pressure exerted on any point in the back of the wall is represented by the ordinate of the triangle of pressure drawn parallel to its base and the whole pressure by the sum of these ordinates, *i.e.*, by the area of the triangle or prism.

When the back of the wall is vertical this area will be $\frac{H^2}{2}$, $H$ being the vertical depth of water and the pressure $\frac{wH^2}{2}$, $w$ being the unit weight of water, one thirty-sixth of a ton per cubic foot.

When the back of the wall is inclined, the pressure will be $\frac{wH_1 H}{2}$, $H_1$ being the inclined height of the back of the wall, which being always greater than $H$, the vertical depth, the latter expression is also greater.

(2) In designing the section of masonry dams and weirs, the following conditions are usually prescribed:—

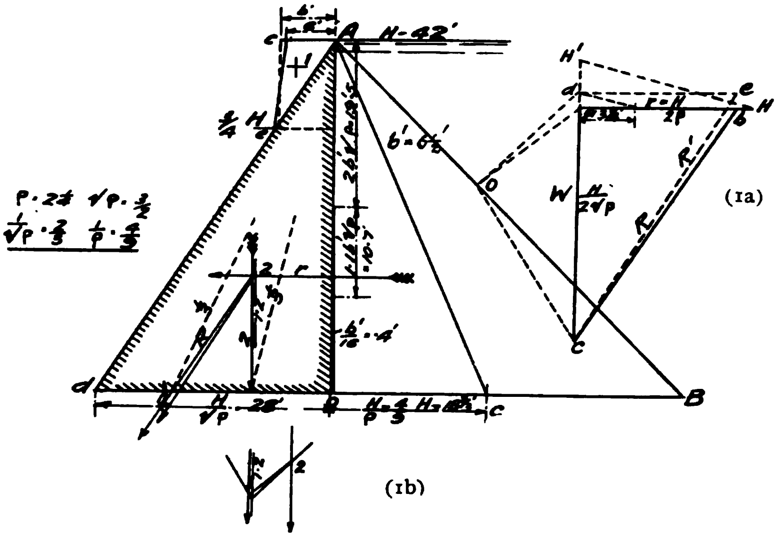
(1) There must be no tension in the masonry.

(2) The maximum pressure on any plane must not exceed a certain prescribed safe limit.

(3) There must be no tendency to slide at the joints.

The first condition is fulfilled by designing the section of such a width that the line of pressures, with reservoir either full or empty, shall fall at or just within the middle third of the base width at any depth. This, in almost all cases, also fulfils the third condition, as the angle of incidence of the resultant on a horizontal plane will always be more obtuse than the angle of friction of the material.

The second proviso is met by spreading out the base beyond what would be required to fulfil condition (2) to ensure that the maximum pressure exerted on a plane normal to the direction of the resultant does not exceed the prescribed limit. In large dams or weirs this limiting pressure is usually fixed at from 8 to 10 tons, but in dams arched on plan, over 20 tons is admissible.


A
H = 42'
c
d
B
O
C
H'
e
H
W
R
R'
(1a)
(1b)
$\rho = 2\frac{1}{4}$
$\sqrt{\rho} = \frac{3}{2}$
$\frac{1}{\sqrt{\rho}} = \frac{2}{3}$
$\frac{1}{\rho} = \frac{4}{9}$
$\frac{H}{\sqrt{\rho}} = 28'$
$b' = 6\frac{1}{2}'$
$\frac{b'}{16} = 4'$
$\frac{H}{2\sqrt{\rho}}$

FIGS. 1, 1a, 1b.

(3) The theoretically correct outline of a dam subjected to water pressure is a right-angled triangle with a vertical back, in which any horizontal ordinate at a depth $H$ below the summit equals $\frac{H}{\sqrt{\rho}}$. In this expression $\rho$ denotes the specific gravity of the material used, which varies from 1·8 to 2 for brickwork and brick concrete, to 2¼ and 2½ and over for rubble-stone and concrete in cement. Assuming a value of $\rho = 2\frac{1}{4}$, which is the most common, the base width, according to the above expression, will be of $\frac{2}{3} H$ and the face batter 1 to 1½.

The hatched triangle $AOd$ in Fig. 1 represents this theoretical outline, which will be designated "the elementary triangular profile."

In the same figure the triangle of water pressure $AOB$ is attached to the back of the profile in which $OB = OA = H$.

In accordance with the procedure already adopted in the case of earth retaining walls, this triangle of pressure has to be reduced to its equivalent area in the unit weight of the dam, which reduction is effected by reducing the base $OB$ to $OC = OB \times \frac{1}{\rho}$, or with $\rho = 2\frac{1}{4}$, $OC$ will measure four-ninths of $H$. In this case $H = 42$ feet, and $OC = 42 \times \frac{4}{9} = 18\frac{2}{3}$ feet.

A graphical test of the accuracy of the elementary profile with regard to condition (1) is effected by drawing its resultant line through the base, for with this triangular section, if its incidence is at the right point at the extreme base, it will be so at any higher base.

(4) The graphical procedure will now be briefly explained. In Fig. 1a the force lines $W$ and $r$ are respectively vertical and horizontal, the direction of the water pressure being normal to the vertical back of the wall, and in length are equal to the half-widths of their respective triangles, which are $\frac{H}{2\sqrt{\rho}}$ and $\frac{H}{2\rho}$ respectively, the height $H$ common to both areas being omitted for simplification. The closing line $R$ or $bc$ represents the resultant in magnitude and direction.

Reverting to the profile, the point of application of the horizontal force $r$, is at the centre of gravity of the triangle of pressure, *i.e.*, at one-third of $H$ above the base line. The line $r$ is drawn through this point parallel to the base of the triangle until it meets the vertical through the centre of gravity of the triangle of masonry, whence the resultant $R$ is drawn reciprocal to $R$ in Fig. 1a. This will be found to intersect the base line exactly at one-third the length of base from the outer toe. Further, the vertical through the centre of gravity of the masonry wall also cuts the base exactly at the inner third division. The former point indicates the incidence of the resultant lines of pressure reservoir full and the latter when empty. The absolute correctness of the profile as regards condition (1) which was arrived at by analytical methods, is thus demonstrated graphically. Condition (2) is also satisfied up to a certain depth when the limiting stress is reached.

Below this point the elementary triangular profile will have to be departed from, the base requiring to be splayed out both fore and aft so as to increase the bearing surface.

(5) This limiting depth with regard to reservoir full is reached when $H = \frac{\lambda}{w(\rho + 1)}$, in which expression, $\lambda$ is the limiting stress; for example, if $\lambda$ be 8 tons per square foot, then $H = \frac{8}{\frac{1}{36} \times \frac{13}{4}} = 88$ feet ($\rho$ being $2\frac{1}{4}$), with $\lambda = 10$ tons, $H = 111$ feet.

With regard to reservoir empty the limiting depth $H, = \frac{\lambda}{\rho w}$.

We thus see that the elementary profile will answer for all dams of less height than about 90 feet, so that the troublesome calculations necessary for determining the base width below the limiting depth are only required in exceptionally high dams. It should be noted that if the back of the triangular profile be inclined from the perpendicular, it will at once cause $R$ to fall outside the proper point, thus demonstrating that, as in the case of retaining walls, the vertical backed dam wall is the most economical form.

(6) The theoretical triangular section will however have to be modified in practice by the superposition of a block of masonry to form the top crest. This top width will have to be of considerable dimensions in countries subject to great alternations of temperature, as the ensuing expansion and contraction of the masonry or concrete necessitates a top width of sufficient thickness to counteract these occult forces without failure of, or undue tension in, the masonry.

No definite rules have hitherto been laid down for the top width of a dam. In "Water Works Engineering" the crest width is stated to vary from 5 feet to 10 feet, and a rule of $\frac{H}{10}$ is suggested. This, however, would be only suitable for high dams of 60 feet to 100 feet and upwards, but not to low dams, so it cannot be used as a universally applicable formula. The advantage of making the top width a fixed fraction of the height is that one profile worked out to this proportion of top width will suit any height of dam, the sections being similar. In Wegmann's treatise model profiles have been calculated out on this supposition. Molesworth's rule is $\cdot 3H$, for top widths of low dams presumably under 25 feet in height, and Courtney, on "Masonry Dams from Inception to Completion," decides on a crest width of $\sqrt{H} + 2$. This latter formula would give a value of $b$ much in excess of requirements, unless in exceptional cases. For general purposes the formula

$$b = \sqrt{H}$$

would seem to be satisfactory for all values of $H$ above a certain limit, besides closely corresponding with actual practice. Thus when

$$H = 50 \quad b = 7$$
$$H = 100 \quad b = 10$$
$$H = 200 \quad b = 14$$

which widths are not in disagreement with many examples. The ratio $b = \sqrt{H}$ will therefore be adopted in this work for dams at and exceeding 50 feet in height. Below this height the proportion of $b = \sqrt{H}$ will make the crest too heavy to be suitable, and, as will be explained later, the dimensions will be gradually reduced as the height decreases.

(7) The imposition of a block of masonry on the top of the triangular profile $AOd$ (*vide* Fig. 1) will modify the direction of the line of pressures, reservoir full or empty, in proportion to its relative size. The influence exerted by the crest in the case of reservoir full, would be to shift the centre of gravity of the wall towards the back, thus tending to bring the line of pressure within the middle third, while under the conditions of reservoir empty it will tend to throw the line outside the middle third.

With a square vertically faced top, as shown by the dotted face line in Fig. 1 (reservoir empty), the line of pressures will begin to diverge outside the middle third towards the back of the wall at a point marked $E$ which is situated at a depth of $2b\sqrt{\rho}$ below the crest.

Below this point $E$, the vertical through the centre of gravity of the section lying above any base will fall inside the inner third of its horizontal joint, gradually approaching the back down to the point *F*, which is situated at a depth of $3{\cdot}1b\sqrt{\rho}$ below the crest *A*. Below the point *F*, the line of pressure would begin to recede from the back, tending to recover its original position. The distances *AE* and *AF* are taken from "Waterworks Engineering," to which work the reader is referred for proofs or further information.

(8) To compensate for this divergence of the lines of pressure from their proper direction, a battered piece has to be added on the back of the wall. This will commence at *E* and end at *F*, from which latter point the new back will proceed in a vertical direction down to the limiting depth when $H = \frac{\lambda}{w(\rho+1)}$.

The width of the extra strip at the point *F* and below, is $\frac{b}{16}$ (with sufficient approximation) the batter from *E* to *F* will be $\frac{1}{18\sqrt{\rho}}$, or 1 in 27. These formulas apply to a rectangular top block, but can be made applicable to one shaped in the full lines of Fig. 1, which has its face inclined. The adjustment is effected by increasing the width of *b*, up to a vertical line halving the batter of the face, which increased value is used in the formula. This is shown by a dotted line.

(9) We have already seen that the effect of the imposition of a wide crested top on the line of pressures of reservoir full acts in favour of the dam bringing the line within the middle third. Where the top is relatively very small to the mass of the dam this divergence can well be neglected, but with the larger proportioned crest with battering faces adopted in this work the effect will be greater, and advantage can be taken of it to strip a piece off the face of the primitive triangular profile, and thus avoid waste of material. This is done in Wegmann's model profiles, though in "Water Works Engineering" the original face line is left undisturbed. The alteration of the face line can best be effected by adopting two or more inclined faces approximating to a curve, thus easing off the abruptness in the junction of the top with the face. The extent of this reduction in section can best be effected by tentative graphical methods, different dispositions being successively tried till a correct, or approximately correct, solution is obtained. This operation appears more troublesome than it really is. The divergence of the resultant of the forces at the base line within the middle third will form an excellent guide in deciding how much material can be removed from the face, so that seldom more than one trial will be found necessary.

As will be seen later, practical rules will be deduced for the delineation of profiles of various heights, which will greatly simplify the process of design. The foregoing applies only to dams of a height less than the limiting depth, *i.e.*, less than 90 to 110 feet, roughly, the latter depending on the value of the limit stress imposed on the masonry, and of the specific gravity of the material employed.

This reduction in the face will affect the position of the centre of gravity of the wall, and consequently the line of pressure, reservoir empty. To allow for this, the width of the back strip added on account of the widened crest will have to be further slightly increased.

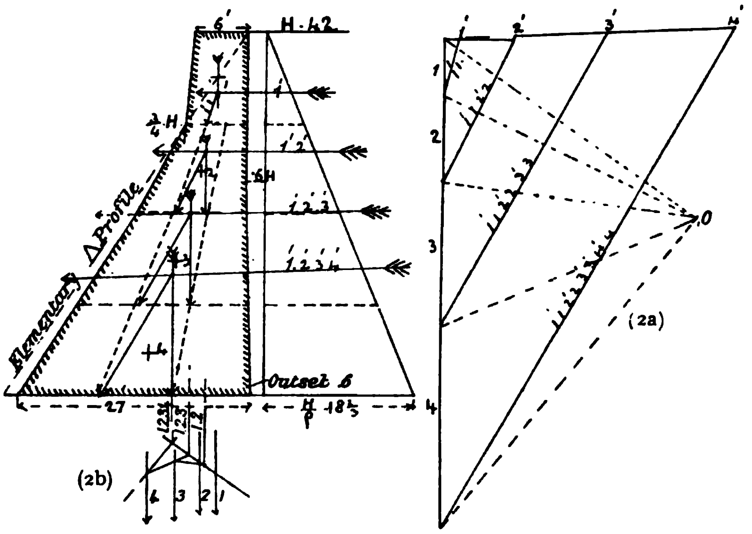
**(10)** We will now proceed to mark the resultant line of pressure on the base of the elementary section, including the imposed crest.

In Fig. 1a the original triangle of forces *abc* can be utilised in part by adding *ad*, equalling the half-width of the superimposed triangular piece *ACe* in Fig. 1, reduced to the base $H$, which is the multiplicand of $r$ and $W$ in the original force triangle *abc*. The height *ce*, Fig. 1, is $\frac{H}{4}$, hence the additional length *ad* equalling $\frac{1}{4} \times 3\frac{1}{2}$, is added to the original load line *ac*, Fig. 1a. As $r$ is the same in both cases, the line *de*, Fig. 1a, parallel to, and equal with *ab* will represent this force, the closing line *ec* or $R_1$ completing the triangle of forces. The next step is to find the centre of gravity of division (1), viz., of the triangle *Ace* in Fig. 1, that of *AOd* or (2) being known from the previous working. The centre of gravity of these two forces is now found according to the usual method by the force and ray polygon shown to left of $W$ in Fig. 1a, and the reciprocal funicular polygon in Fig. 1b. The vertical resultant marked 1, 2 thus obtained, is now projected on the profile (dotted) till it meets the feathered force line $r$. This intersection is the starting point for $R_1$ while that of the same vertical with the base line *Od*, gives the position of the centre of pressure on the base (reservoir empty). Similarly the intersection of $R_1$ with the base line is the centre of pressure on the base with reservoir full. Inspection of these points show the effect of the imposition of the crest triangle; $R_1$ now falls nearly 1 foot within the middle third, while $W_1$ is a trifle outside. To remedy this, the narrow extra strip added on at the back will bring $W_1$ into its proper proportional position, and also on the other side, the base width can be reduced to a greater extent by bringing the outer toe in, thus taking a strip off the face line of the profile.

**(11)** The amended profile of Fig. 1, in which the face line of the elementary triangle is altered into a new one, recessed within the former, and composed of two inclined surfaces, is illustrated in Fig. 2. At the back, the battered and vertical portions of the added strip of a width of $\frac{b}{16}$, are combined in Fig. 2 into one continuous batter, starting about the point $E$ at $\cdot 6H$ above the base, and projecting beyond the vertical about $1\frac{1}{2} \times \frac{b}{16}$, or $\cdot 6$ feet. This simplifies the section without appreciably impairing the correctness of the outline.

The revised outline of the face resembles that adopted by Wegmann in his theoretical type No. 11, Plate XV., for a 200 feet dam. The base width as measured from the altered position of the inner toe is taken as 27 feet, and the face line is formed of two planes, the lower drawn from the toe, parallel to the triangular profile to a height of $\frac{H}{2}$, where a second plane joins the extremity with the root of the crest at $\frac{3H}{4}$.

(**12**) The procedure of drawing the line of pressures in Fig. 2 is that usually adopted for showing the points of pressure reservoir empty and full. The system consists of dividing the profile and triangle of water pressure into a certain number of equal laminas, in this case 4, numbered accordingly 1 to 4, while the corresponding divisions of the triangle of pressure are numbered $1_1$ to $4_1$. This latter is given a horizontal base, and all half-widths will be measured parallel to it. The inclination of the back, being trifling, is neglected, except in the directions of the force lines 1, 2, etc. A series of independent combinations are now formed in the force triangles in Fig. 2a, viz., of $1_1$ and 1; $1_1$, $2_1$, with 1, 2; $1_1$, $2_1$, $3_1$, with 1, 2, 3 and $1_1$, $2_1$, $3_1$, $4_1$, with 1, 2, 3, 4. The centres of pressure of these three last combinations are discovered by use of the funicular *2b*, and the intersection of the projected vertical resultants with the horizontal and inclined forces of each combination. The intersection of those same verticals, with their respective base lines, give points on the line of pressure, reservoir empty. Thus each pair of forces is independently dealt with; this arrangement is so far advantageous in that error is not perpetuated.



FIGS. 2, 2a, 2b.

(**13**) In the cases illustrated in Figs. 1 and 2, the back has been assumed as vertical; when it is inclined, according to the laws of hydrostatics, the fluid acts normal to the inclined back of wall. This is exemplified in Fig. 3, where the triangle *AOa* represents the actual area of water pressure. Here

*AO* is greater than *H*, and the triangle *AOa* is not equilateral, *AO* being equal to *H* × sec. $\theta$, $\theta$ being the angle of inclination of the back *AO* to the vertical. In graphical processes it is most convenient to avoid calculation of areas if possible, and to represent them by half-widths of triangles or trapeziums. This has been invariably done in Chap. I. when dealing with retaining walls. To effect this, the wall and the triangle of pressure must have the vertical height *H* in common, so that it can be eliminated from both sets of forces. In the case illustrated in Fig. 3 it is practicable to convert the triangle of pressure *AOa* into another having a horizontal base and also containing *H* as perpendicular height. This is effected by drawing *ab* parallel to *AO* intersecting the horizontal base line at *b*, then *AOb* will be the triangle required. Its area is equal to *AOa*, both the triangles having the same base *AO* and lying between the same parallels *AO* and *ab*. It further has *H* for perpendicular height, counting *Ob* as its base, and any triangle with apex on the water surface line, and the same base *Ob* will clearly be equal to *AOb* and also *AOa*.

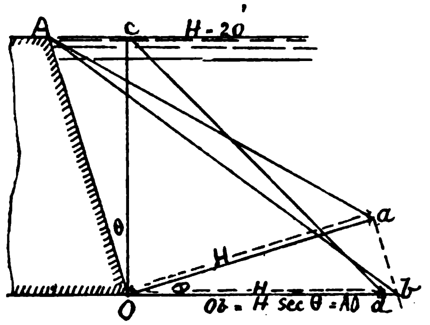


FIG. 3.

The new base *Ob* is clearly equal to *OA*.

The procedure now will be exactly similar to that already described in the case of retaining walls. Horizontal ordinates parallel to the base *Ob* will represent the pressure at each point on the back of the wall. They will differ from the ordinates of the true triangle of pressure *AOa*, but the final resultant when multiplied by *H* will be the same as the ordinates of *AOa*, multiplied by its vertical height *AO*, the two triangles being equal in area.

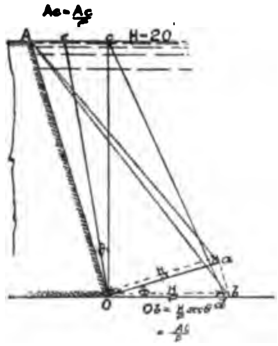


FIG. 4.

(14) The triangle *AOa* is composed of a combination of vertical and horizontal forces. To decompose these into separate areas, make *Od* on the base line equal to *H*, and draw the vertical *cO*, then *AcO* represents the area of water resting on the back *AO*, and the triangle *cOd* lying right of the vertical plane, *cO* represents the horizontal pressure.

It should be noticed that *Od* = *Oa* is not the horizontal component of *Oa*. Water being a fluid, the pressure is never less than that of the unit depth. Thus at a depth of *H* from surface the horizontal unit pressure is always equal to *H*. If inclined, it means that vertical forces are combined with the horizontal. The inclined triangle *AOa* is, of course, greater than *cOd*.

In Fig. 4, the triangles of pressure are reduced to an equivalent weight of masonry, having a specific gravity of 2½. The wedge of water lying on the back of the wall $AcO$ is here represented by the triangle $AeO$, $Ae$ being $Ac \times \frac{1}{\rho}$. The base $Ob = \frac{AO}{\rho}$.

**(15)** The weak point in the graphical process described in par. 12 is that the forces $r$ are drawn normal to that part of the back on which they happen to impinge; thus the final for the whole wall $1_1$, $2_1$, $3_1$, $4_1$, is normal to the inclined lower part, not to the whole, whereas also the direction of $1_1$, $2_1$, is ambiguous, as the back of the wall has two different inclinations in the lamina $1 + 2$. It thus follows that this system is wanting in absolute accuracy. In Fig. 2 the battered portion is but slight, and the scale small, so that practically the inaccuracy is inappreciable; but for absolute precision Haessler's new system is the only one altogether suitable.

In this system the groups of forces are not taken as separate entities, but are independent, the directions of the inclined forces $r$, being normal to the back of each lamina 1, 2, 3, and 4, instead of, as in the old system, applying to a combination of several such laminas, each having a possible different inclination to the vertical. With a uniformly battered back, the old system is quite suitable, but when the base is formed of several planes the Haessler system is distinctly preferable. Haessler's system is exhibited in Fig. 16, Chap. I.

**(16)** From the foregoing it will now be possible to devise a practical system of design for dams of varying heights closely approximating to the exact section required by theory.

The first point for determination is a definite rule for the top or crest width. It has already been noticed in par. 6 that the width $\sqrt{H}$ is too top heavy for values of $H$ much lower than 50 feet. It is further deemed advisable that the face line of the superimposed crest should have an outward batter, and should join the hypothenuse of the elementary triangular profile at a point not higher than $\frac{3}{4}H$. The thickness of the wall at this level will therefore be invariably one quarter the base width of the elementary triangle. This arrangement ensures a not too abrupt junction with the lower part of the face, and forms a wide base for the upper fourth of the wall. The matter of face batter and solidity in the top of a dam is strongly insisted on in "Courtney on Dams," which the author considers to be a sound view, and preferable to the light vertically faced top crest often adopted, which is not suitable in sub-tropical climates.

The following rules would then appear to meet the case in a satisfactory manner :—

Width of crest for $H = 50$, or above, $\sqrt{H}$, measured from the apex of the elementary triangle which forms the inner line of crest. The outer face will be formed by a line joining the outside of crest thus obtained with the point of intersection of $\frac{3}{4}H$ level with the hypothenuse of the elementary triangle. When $H$ is below 50 feet, the face will batter at 1 in 10 above this point, *i.e.*, from $\frac{3}{4}H$, thus fixing the top width, which will be base width $\times \frac{H}{40}$, or if the base width is $\frac{H}{\sqrt{\rho}}$ the top width will become $\frac{1}{4}\left(\frac{H}{\sqrt{\rho}} - \frac{H}{10}\right)$.

When $H$ 50 and $\rho=2\frac{1}{4}$ this expression will equal 7 feet, the $\sqrt{50}$ is likewise 7 feet or very near it. Hence when $H$ equal 50 and for all greater values, the top width will become $\sqrt{H}$, below this height the other formula is applicable.

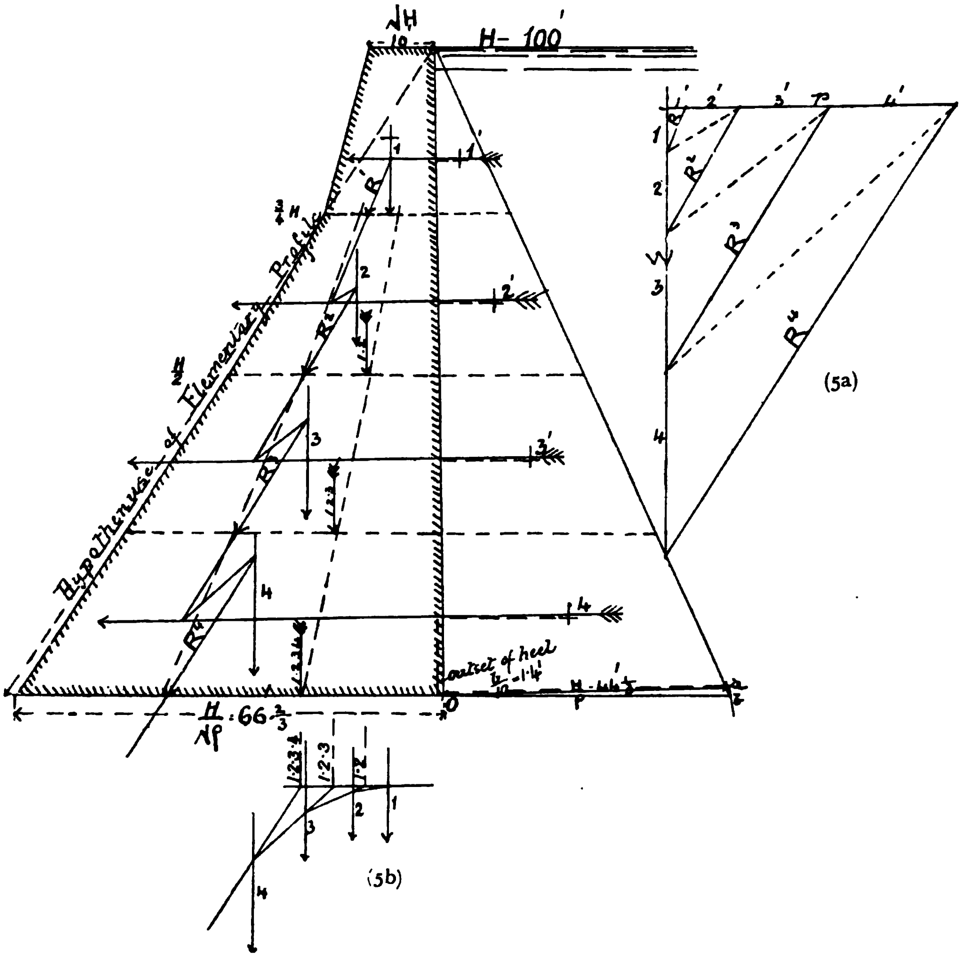
**(17)** Now as regards the back batter. This can be formed as in Fig. 1, of two vertical portions connected by a battered part, the extra thickness imparted to the base being $\frac{b}{16}$ ($b$ being the mean imposed top width). This back line is strictly accurate, if the hypothenuse of the elementary triangular section be retained for the face line of the profile below $\frac{3}{4}H$, as recommended by Dr. Tudsbery in "Water Works Engineering." The retention of this line as a face line will, however, make the base greater than $\frac{H}{\sqrt{\rho}}$ and there will be an excess of material on this side sufficient to appreciably influence the intersection of $R_1$, Fig. 1, which will fall within the middle third. With the heavier top width adopted by the author, this surplusage of material in the face becomes more accentuated, particularly in dams of moderate height, say below 80 feet, consequently it is deemed advisable to reduce the section somewhat by bringing the face line in. This action will have its effect on the back line, as removal of material from the face will alter the centre of gravity so that $\frac{b}{16}$ will be a hardly sufficient extra width to the back, to bring the centre of pressure, reservoir empty, exactly on to the inner third division of the base. The difference will, however, be very small. If the back of the wall be built in a continuous batter, from the top, as is done in Figs. 5, 6 and 7, the profile will be much simplified and the structural advantages of a batter in breaking joint, a property particularly valuable where rubble masonry is concerned, would be also secured. If the base of this additional triangular strip were made $\frac{b}{10}$ it would satisfy all requirements. In this, $b$ is not the actual crest width, but the mean thickness of the upper fourth part of dam.

**(18)** As regards the base width, that of $\frac{H}{\sqrt{\rho}}$, the same as the base width of the triangular profile, has been adopted. It gives a thickness somewhat in excess of actual requirements, as can be seen from the position of the centre of pressure in the base in Figs. 5 to 7, but as these profiles and formulas are given as a guide to the designer, who from the data thus supplied can rapidly design a section of the exact proportions required, absolute precision, which would cause the formula to be complicated, is rejected for the sake of simplicity.

**(19)** In Fig. 5, the profile is delineated in accordance with the rules adopted.

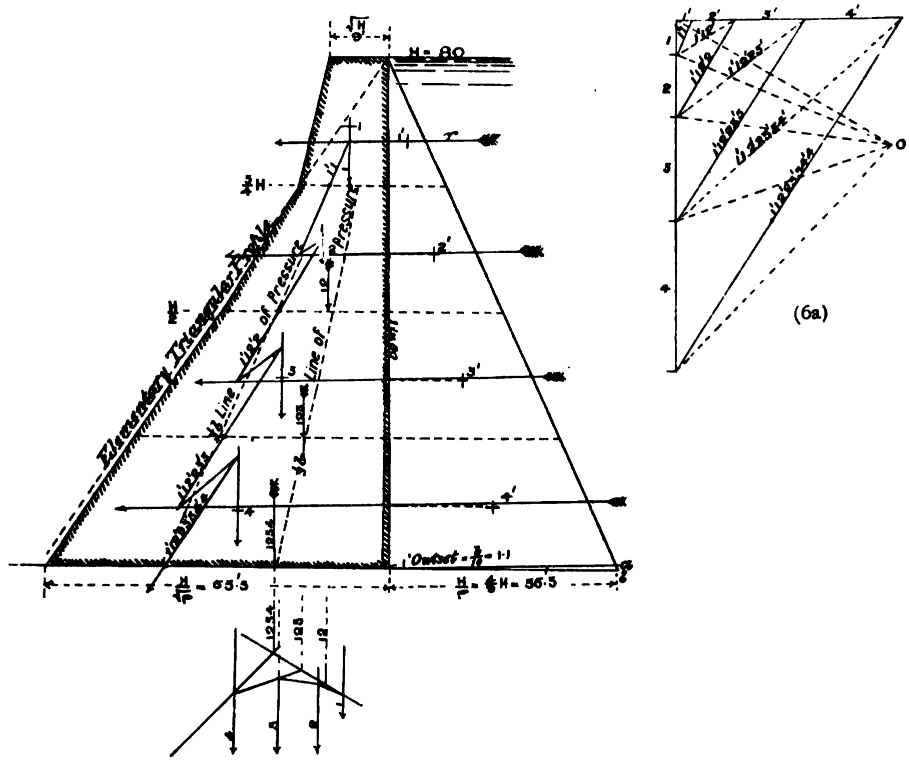
The height of $H$ is 100 feet and the base width is two-thirds of 100, or $66\frac{2}{3}$ feet. The outset at the heel is made $\frac{b}{10}$, or 1·4 feet. The face line at level $\frac{3}{4}H$

coincides with the hypothenuse of the elementary triangle, whence upward a line connects this point with the outer edge of the crest, which is 10 feet, *i.e.* $\sqrt{H}$ in width, while downwards another line joins at level $\frac{H}{2}$ the extremity of a line drawn from the toe parallel to the hypothenuse of the elementary triangular profile.



FIGS. 5. 5a, 5b.

The section of the wall, as also the triangle of water pressure, is divided into four equal laminas of 25 feet. The weights of these laminas can therefore be represented in the force polygon Fig. 5a by their respective half-widths, the height common to each lamina, viz., $\frac{H}{4}$ being eliminated as a common factor. Haessler's system is used in the reciprocal lines on the profile, but the inclinations of the different divisions of *r* being the same, there being a uniform back batter, these forces can be placed in one



(6a)

(6b)

Figs. 6, 6a, 6b.

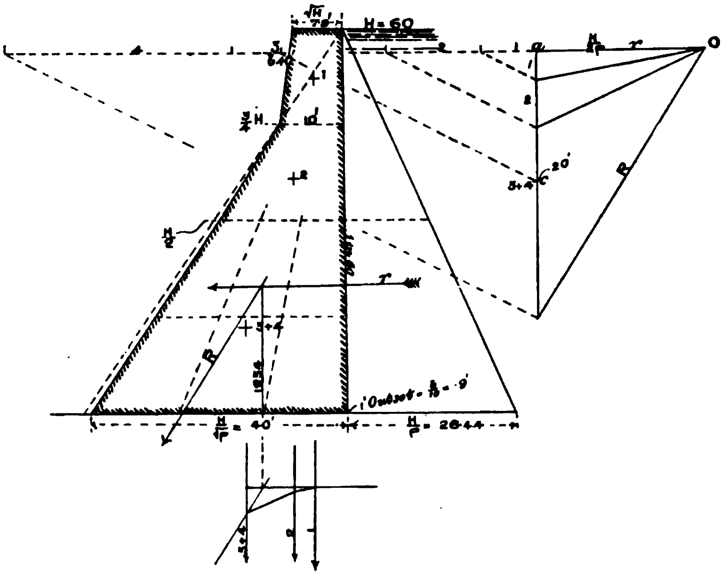


Fig. 7.

line as shown in Fig. 5a. The lines of pressure of $R$, reservoir full, fall at or just within the middle third at the base of each lamina.

The incidences of $W$, the vertical weights when reservoir is empty, are arrived at by the construction of the funicular polygon 5b with equally satisfactory results.

The actual value of $R_4$ in tons is obtained by measuring its length on Fig. 5a, multiplying this quantity by 25 (the value of $\frac{H}{4}$ eliminated as a common factor) and the quotient again by $w\rho$ or $\frac{1}{36} \times 2\frac{1}{4} = \frac{1}{16}$.

(**20**) In Figs. 6 and 7 similar examples of design are given. In the former case $H$ is 80 feet, and in the latter 60 feet.

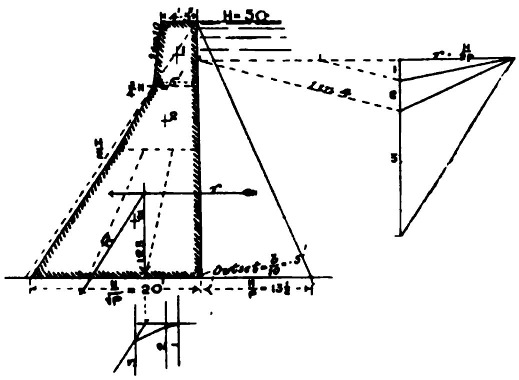


FIG. 8.

(**21**) In Fig. 8 the value of $H$ is 30 feet and in Fig. 9 20 feet. In both these cases the top width is less than $\sqrt{H}$, being formed by drawing a line upward from the point coinciding with the hypothenuse of the elementary profile at $\frac{3}{4}H$, at an inclination with the vertical of 1 in 10, which line will determine the width of the crest. In Fig. 9 the double batter of the face below $\frac{3}{4}H$ is converted into a single line joining the toe, the saving in the adoption of the double slope common to all profiles from Figs. 5 to 8 being too trifling for consideration, and the back, for similar reasons, is made vertical.

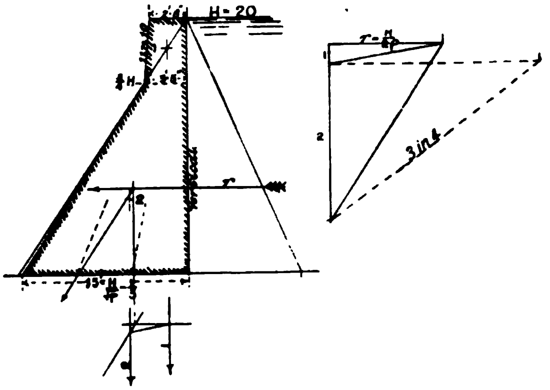


FIG. 9.

(**22**) The graphical system which is almost exclusively made use of in this work to decide statical problems is in most cases infinitely easier of application than the analytical, and although naturally lacking in the close mathematical accuracy which alone can be obtained from trigonometrical or arithmetical methods, yet is quite sufficiently precise for all practical requirements. The graphical system is peculiarly valuable for the purpose of tentative design, as it exhibits at once to the eye the proportions that require alteration or adjustment. Theoretical profiles calculated to decimals of a foot are never rigidly adhered to in practice, and the analytical equivalent to what is equally well obtained by easy graphical methods involves the use of complicated formulas which are laborious to work out.

(**23**) The foregoing results will now be tabulated for easy reference.

### *Rules of Design for Low Dams, i.e., below* 100 *feet.*

Crest width, at and over 50 feet $= \sqrt{H}$

" below " $= \frac{1}{4}\left(\frac{H}{\sqrt{\rho}} - \frac{H}{10}\right)$

Back batter is formed by a back set at heel beyond vertical through inner crest $= \frac{b}{10}$ ($b$ = middle width of top quarter of dam).

Base width $= \frac{H}{\sqrt{\rho}}$.

Face batters: Upper quarter, base thickness one-quarter $H$, measured from the vertical drawn through the inner edge of crest. Lower half, batter 2 in 3 up from outer toe of base. Third quarter, face line joins last two extremities.

### *Distribution of Pressure on Foundations.*

(**24**) Before proceeding to undertake the description of method pursued in designing the base of dams below the limiting depth, it will be as well to investigate the distribution of pressure on the foundations, which is necessary in all large masonry walls and particularly so in dams where the maximum pressure, reservoir full and empty, has to be ascertained. This applies also to walls retaining earth; in such cases, however, the real position of the centre of pressure at the base is not precisely known, owing to the imperfect nature of the statistics on which the theory of earth pressure is based, the important factor cohesion, which acts entirely as an asset to the resistance of the wall, having to be omitted owing to our inability to fix its precise value. Thus the position of the centre of pressure on which the design of the wall is based, appears to be in a more unfavourable situation than what it actually would occupy, were credit given to the unknown force, viz., cohesion, which acts in favour of the stability of the wall. Consequently the maximum pressure in the masonry would likewise be exaggerated. Retaining walls, however, in irrigation works, are rarely of such dimensions as to require any special investigation into the distribution of pressure on the foundation.

The formula already given of $H = \frac{\lambda}{w(\rho + 1)}$, for limiting depth of the elementary triangular profile, is based on the theory that the maximum pressure in the masonry of a wall occurs on a plane at right angles to the direction of the resultant, *i.e.*, to the hypothenuse of the elementary triangle. When this plane is projected on to the horizontal base it causes the maximum intensity to be increased in proportion as the base width is in excess of this plane.

**(25)** In Fig. 10, a replica of part of Fig. 5 and 5a, $R$ is the resultant, but the maximum intensity of the pressure on the base is not measured by $R$ but by the hypothenuse $N_1$. In Fig. 10a the triangle of pressure is *abc* or *WrR*. If *be* be drawn vertically, and *ce* at right angles to $R$, and *cd* horizontally, then the line *bd* or $N$ represents the vertical component of $R$, which, as $r$ is horizontal, is equal to $W$, but the line *be* or $N_1$ represents the maximum pressure as distributed on the base. Consequently in estimating the distribution of pressure in the masonry on the base of a wall, $N_1$ has to be considered as representing the resultant, not $R$, although the point of application is that of $R$.

This force $N_1$ can be considered as if acting vertically on the base, although actually it is an inclined force. On the other hand, in estimating the distribution of weight on the foundations of a wall the vertical component of $R$, viz., $N$, should be considered, the inclined pressure being absorbed in the masonry of the wall, and by frictional contact with the ground.

**(26)** In Fig. 10 the base of the wall is *ad* or $b$; the centre pressure $R$ intersects this base at the point $e$. Let the distance *ef*, *i.e.*, from the point $e$ to $f$, the centre of the base, be termed $c$, the total inclined force acting at $e$ being $N_1$. Then the mean pressure on the base will be $\frac{N_1}{b}$. If the pressure acting at the toe of the wall be termed $P$ and that of a contrary sense acting at the heel be termed $P_1$, then the maximum intensity either compressive or tensive is expressed by the following formula:—

$$P = \frac{N_1}{b}\left(1 + \frac{6c}{b}\right) \text{ and } P_1 = \frac{N_1}{b}\left(1 - \frac{6c}{b}\right).$$

**(27)** When the point $e$ falls at the extreme toe $a$, the fraction $\frac{c}{b}$ will equal one-half, whence $P$ will equal $\frac{N_1}{b}(1 + 3) = 4\frac{N_1}{b}$, and $P_1$ will equal $\frac{N_1}{b}(1 - 3) = -2\frac{N_1}{b}$.

Thus with the incidence of $R$ at the extreme toe, the maximum compression will be at this point four times the mean pressure, and the maximum tension at the heel will be twice the mean pressure.

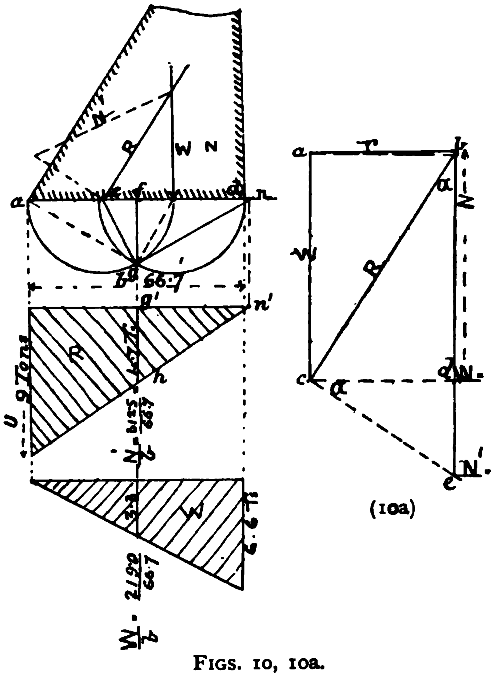
In the same way when $c$ is at the second third division of the base counting from the heel,

$P$ will equal $2\frac{N_1}{b}$ and $P_1$ will be zero.

When $e$ is at the centre point of the base, $c$ becomes zero; consequently $P = \frac{N_1}{b}$ and $P_1 = \frac{N_1}{b}$. There is therefore an even distribution of pressure in a plus or compressive sense, equal to $\frac{N_1}{b}$.

**(28)** The graphical method of ascertaining the distribution of pressure is shown in Fig. 10. Two semi-circles are struck on the base, with centres at the two-third divisions, and with radius of $\frac{b}{3}$. From $e$, the point of incidence of $R$, the line $eg$ is drawn to $g$, the intersection of the semi-circles. Again from the point $g$ a line $gn$ is set off at right angles to $eg$, cutting the base, or its continuation at the point $n$. This point $n$ is the antipole of $e$, or the neutral point at which the pressure is nil in either sense.

Below the profile another projection of the base is made. From $g$, a perpendicular is let fall cutting the new base line in $g_1$, and its continuation $g_1h$ is made equal to $\frac{N_1}{b}$, or to the mean pressure. Another perpendicular is then let fall from $n$, cutting the base in $n_1$, and $n_1h$ is joined and continued till it meets the perpendicular from the toe of the base, another line being drawn similarly from the heel of the base. The hatched trapezium thus enclosed represents the distribution of pressure. Thus in Fig. 10, $N_1$ scales 200 nearly. To reduce this to tons, as has already been explained with regard to $R_4$ Fig. 5a, it has to be multiplied by 25 feet, the common depth of the laminas, and by $wp$ or $\frac{1}{36} \times 2\frac{1}{4}$. $N_1$ will therefore be $200 \times \frac{1}{36} \times \frac{9}{4} \times 25 = 312{\cdot}5$ tons, and $\frac{N_1}{b}\ \frac{312{\cdot}5}{66{\cdot}7} = 4{\cdot}67$ tons.



FIGS. 10, 10a.

In Fig. 10 the centre line $g_1h$ is made equal to $\frac{N_1}{b}$, or 4·7 at any scale, and $n_1h$ joined and continued to the toe boundary. Then the hatched trapezium enclosed between the verticals at the toe and heel of the base, the projected line itself and the inclined line $n_1h$ represents the area of pressure on the base, the intensity at each point being measured by the vertical ordinate at that point.

The maximum pressure at the heel $U$ measures 9 tons. In similar manner the distribution of pressure due to $W$, reservoir empty, is shown on the lower diagram.

**(29)** In Fig. 11 the distribution of pressure on the base, due to the incidence of $R$, first at the toe, secondly at the two-third point and thirdly at the centre, is illustrated. In the first case ($R_1$) it will be seen that the neutral point $n$ falls at the first third point. Thus two-thirds of the base is in compression and one-third in tension, the maximum in either case being clearly proportionate to the relative distance of the neutral point from the toe and heel of the base, the compression at the toe being four times, while the tension at the heel is twice the mean pressure or $\frac{N_1}{b}$.

In the second case $R$ intersects at the two-third point, and the consequent position of $n$ is exactly at the heel. The whole base is then in compression and the maximum double the mean.

In the third case ($R_2$) the line $gn$ will be at right angles to $fg$, which is vertical, and consequently is horizontal, and the position of $n$ is indefinite; the area of pressure thus becomes a rectangle with a uniform pressure of $\frac{N_1}{b}$.

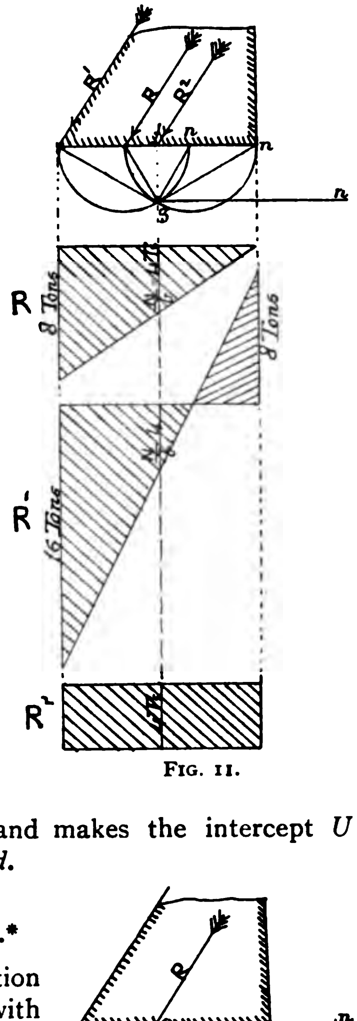


FIG. 11.

**(30)** In Fig. 12 an intermediate case is exemplified, similar to Fig. 10, but more accentuated.

It will be noticed in Fig. 10 that $e$, the point of application of $R_1$, lies somewhat within the middle third; this causes the antipolar point $n$ to fall without the heel $d$, and makes the intercept $U$ less than it would be if $n$ were coincident with $d$.

*Design of Dams below the Limiting Depth H.**

**(31)** We have seen in par. 5 that the section of a dam can be carried down in accordance with the elementary triangular section or with such modifications of it as are advisable, until the depth has a value of $\frac{\lambda}{w(\rho + 1)}$, after which the procedure becomes more complicated, the maximum stress on the masonry, which must not be exceeded, adding another factor to the problem. Not only has the line of pressure reservoir full or empty to fall at the middle third, but the width will have to be increased in a greater ratio to satisfy condition (1). This could be effected by means of trial and error

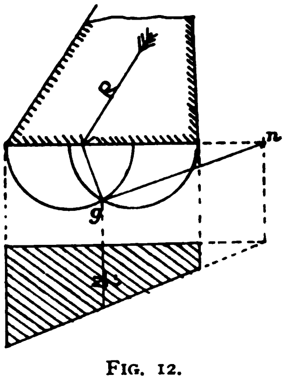


FIG. 12.

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* In recent American practice in the Roosevelt arched and New Croton straight dams, the elementary profile with base $= H \div \sqrt{\rho}$ is retained to full depth without increase, the maximum unit stress being 20 tons and over.

by graphical process, but would be very troublesome, and figured calculation from formulas deduced by analytical methods will be found easier to use. This difficult subject has been very ably treated in the "Principles of Water Works Engineering," and an excellent example of the method of working out the varying width is given in this work, which will be reproduced below in a condensed form.

The solution of this problem is based on the following:—If a rigid body rest on a horizontal surface, the distance of the centre of pressure from the extremity of the base at which the maximum stress occurs is—

$$\frac{b}{3}\left(2 - \frac{s}{2s_1}\right) \quad (1)$$

where $b$ is the width of the base, $s$ the maximum stress intensity in tons, the average pressure being $s_1$, from which the following formula giving values of $b$ is deduced:—

$$b = \frac{wh^3}{\lambda}\left(1 + \frac{w^2 h^4}{4N}\right) \quad (2)$$

$\lambda$ the limiting and $s$ the maximum stress being in this case identical. Here $w$ = weight of water per cubic foot in tons, viz. $\frac{1}{36}$ ton, and $H$ = depth of water. $N$ = the vertical forces, viz., the weight of the masonry wall and of the water over the inner face where it is inclined.

From this equation $b$ is found readily with a high degree of accuracy if $N$ be known even approximately.

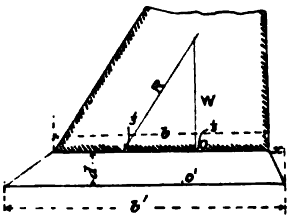


FIG. 13.

(32) Having found $b$, or $b_1$, the next step is to ascertain how much of it must be under the inner face of the dam, *i.e.*, within the vertical through the crest, in order to bring the incidence of the resultant weight of the whole superincumbent mass of the masonry and water to a distance of $\frac{b}{3}$ from the inner toe. This is arrived at by equating the moments of the vertical forces about $O_1$, the point in question (*vide* Fig. 13), to zero, by which means we obtain the following equation, the solution of which gives the value of the distance $x_1$, *i.e.*, the projection of the base at heel of wall:—

$$\left.\begin{aligned}&\frac{pwd}{24}\left(3b^2 - b_1^2 + 6x_1(b + b_1) + 2bb_1\right) - \frac{wx_1}{12}(H + H_1) \times \\ &\qquad (2b_1 - 3x_1) - N\left(\frac{b_1 - b}{3} - x_1\right) = O\end{aligned}\right\} \quad (3)$$

*vide* "Water Works Engineering," p. 222. $H$ is depth of water at base $b$ and $H_1 = H + d$, $d$ being depth of lower lamina.

This formidable-looking quadratic equation is not so difficult to work out as one would judge from its inordinate length.

From formula (2) the value of $b_1$ is obtained, *i.e.*, the bottom width of the first strip to be added below the base, and from (3) $x_1$ is found, which fixes the position of the new lamina with regard to the base above.

(**33**) The following illustration of the practical working out of the formulas (2) and (3), which is a nearly literal transcript from "Water Works Engineering," is given below.

Fig. 14 represents the section of a dam, the value of $\rho$ being $2\frac{1}{4}$ and $\lambda$ 10 tons, whence $H$, the limiting depth, will be equal to 111 feet.

The base width at this depth will be equal to $x + 74{\cdot}0 = 74{\cdot}6$, or, excluding the back strip, 74·0. The face line corresponds with that in the primitive triangular profile, no deductions having been made.

Now the weight of the section, including that of the water overlaying the portion $EF$ of the back, is nearly 265 tons. The successive values of $H$ to be considered will be :—

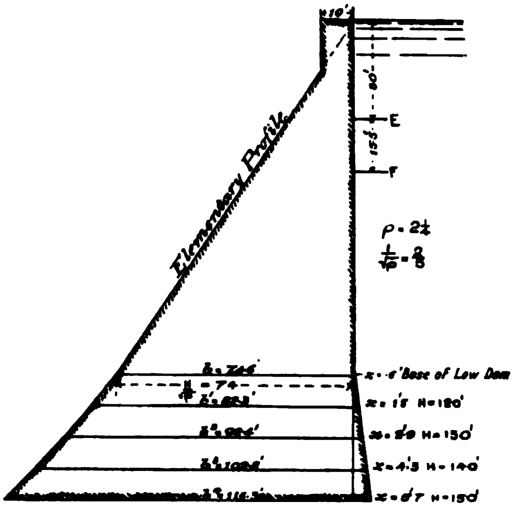


FIG. 14.

| | | |
|---|---|---|
| From crest to base of low dam | - - - | $H$ 111 feet. |
| ,, ,, 1st lamina | - - - | $H_1$ 120 ,, |
| ,, ,, 2nd ,, | - - | $H_2$ 130 ,, |
| ,, ,, 3rd ,, | - - - | $H_3$ 140 ,, |
| ,, ,, 4th ,, | - - - | $H_4$ 150 ,, |

The respective values of $b$ and $x$ (Fig. 13) may be conveniently designated by $b$, $b_1$, $b_2$ and $x$, $x_1$ and $x_2$; also the total weight of the dam and superincumbent water may be by $N$, $N_1$ and $N_2$.

*First Lamina.*—At the base of the low dam (Figs. 13 and 14)

$$N = 265 \text{ tons}, H = 111 \text{ feet}, b = 74{\cdot}6 \text{ feet}, x = O.$$

If the profiles above $b$ be produced down to the level $H_1 = 120$ feet, *i.e.*, 9 feet deeper, the weight of the trapezoid of masonry thus added is

$$\frac{9 \times 74{\cdot}6\left(1 + \frac{120}{111}\right)}{2 \times 16}, \text{ say equal to 44 tons.}$$

Then, as a first approximation,

$$N_1 = 265 + 44 = 309 \text{ tons.}$$

By equation (2), par. 31, introducing the proper value of $\lambda$, or $s$ of 10 tons

$$b_1 = \sqrt{\frac{(120)^3}{360}\left(1 + \frac{(120)^4}{5184 \times (309)^2}\right)}$$
$$= 82{\cdot}5 \text{ feet.}$$

The next step is to find $x_1$ corresponding with this value of $b_1$.

By equation (3), par. 32, we have

$$\frac{9}{16 \times 24}\left[3 \times 5565 - 6806 + 6\,(157{\cdot}1)\,x_1 + 2 \times 6154\right]$$
$$-\frac{x_1}{36 \times 12}\left[231\,(165 - 3x_1)\right] - 265\,(2{\cdot}63 - x_1) = 0;$$

that is
$$1{\cdot}6x_1^2 + 199x_1 - 177 = 0;$$

therefore
$$x_1 = \frac{-199 + \sqrt{39{,}601 + 1133}}{3{\cdot}2},$$
$$= \frac{3}{3{\cdot}2} = 0{\cdot}9 \text{ foot.}$$

The values of $b_1$ and $x_1$ thus found, enable us to obtain a closer approximation to the weight of the trapezoid under consideration, and to determine the weight of water overlying its inner face. The sum of these two quantities may be found to be 47 tons.

Then a second approximation gives

$$N_1 = 265 + 47 = 312 \text{ tons.}$$

Introducing this value of $N$ into equation (2), par. 31, we find, as a second approximation,

$$b_1 = 82{\cdot}3 \text{ feet};$$

which value of $b_1$ is so nearly that previously used in applying equation (3) as not to affect the already found value $x_1 = 0{\cdot}9$ foot.

Thus we have at the base of the first lamina below the low dam—

$$N_1 = 312 \text{ tons, } H_1 = 120 \text{ feet, } b_1 = 82{\cdot}3 \text{ feet, } x_1 = 0{\cdot}9 \text{ foot.}$$

*Second Lamina.*—Proceeding as before, produce the profiles immediately above $b_1$ down to the level $H_2 = 130$ feet; the weight of the trapezoid of masonry thus added, together with that of the water overlying its inner face, is

$$\frac{10}{16}\left[82{\cdot}3 + \frac{1}{2}\cdot\frac{10}{9}\,(82{\cdot}3 - 74{\cdot}6)\right] + \frac{120 + 130}{2 \times 36} \times 1{\cdot}0 = 58 \text{ tons.}$$

Then, as a first approximation,

$$N_2 = 312 + 58 = 370 \text{ tons.}$$

Hence
$$b_2 = \sqrt{\frac{(130)^3}{360}\left(1 + \frac{(130)^4}{5184\,(370)^2}\right)}$$
$$= 92{\cdot}5 \text{ feet.}$$

Applying equation (3), par. 32, and introducing these values of $b_2$ and $N_2$, we find

$$x_2 = 1{\cdot}4 \text{ feet.}$$

Correcting the weight of the trapezoid and its superincumbent water for this value of $x_2$, we have as a second approximation

$$N_2 = 312 + 59 = 371 \text{ tons},$$

whence a re-application of equation (2), par. 31, gives the corrected value

$$b_2 = 92{\cdot}4 \text{ feet}.$$

Thus, at the base of the second lamina,

$$N_2 = 371 \text{ tons}, H_2 = 130 \text{ feet}, b_2 = 92{\cdot}4 \text{ feet}, x_2 = 1{\cdot}4 \text{ feet}.$$

*Third Lamina.*—Produce the profiles above $b_2$ down to the level $H_3 = 140$ feet.

As a first approximation

$$N_3 = 371 + 67 = 438 \text{ tons}.$$

$$b_3 = \sqrt{\frac{(140)^3}{360} \quad \frac{(140)^4}{(1 - 5184\,(438)^2)}}$$

$$= 102{\cdot}8 \text{ feet}.$$

By equation (3), par. 32,

$$x_3 = 1{\cdot}4 \text{ feet}.$$

This introduces no sensible correction in the value of $N_3$; and we therefore have, without further calculation, at the base of the third lamina,

$$N_3 = 438 \text{ tons}, H_3 = 140 \text{ feet}, b_3 = 102{\cdot}8 \text{ feet}, x_3 = 1{\cdot}4 \text{ feet}.$$

*Fourth Lamina.*—Produce the profiles above $b_3$ down to the level $H_4 = 150$ feet.

As a first approximation,

$$N_4 = 438 + 75 = 513 \text{ tons}.$$

$$b_4 = \sqrt{\frac{(150)^3}{360}\left(1 + \frac{(150)^4}{5184\,(513)^2}\right)}$$

$$= 113{\cdot}3 \text{ feet}.$$

By equation (3), par. 32,

$$x_4 = 1{\cdot}4 \text{ feet}.$$

Again no sensible correction is introduced into $N_4$, and we have at the base of the fourth lamina

$$N_4 = 513 \text{ tons}, H_4 = 150 \text{ feet}, b_4 = 113{\cdot}3 \text{ feet}, x_4 = 1{\cdot}4 \text{ feet}.$$

Summarising these particulars, we have:—

| Depth below crest of dam (H). | Breadth of base (b). | Weight of Masonry. | Breadth measuring from axis under inner face (x). | Weight of water over inner face. |
|---|---|---|---|---|
| Feet. | Feet. | Tons. | Feet. | Tons. |
| 0 | 10 | — | — | — |
| 111 | 74·6 | 264 | 0·6 | $\frac{2}{3}$ |
| 120 | 82·3 | 308 | 1·5 | 4 |
| 130 | 92·4 | 362 | 2·9 | 9 |
| 140 | 102·8 | 424 | 4·3 | 14 |
| 150 | 113·3 | 493 | 5·7 | 20 |

(**34**) The foundations of dams are often carried for a great depth below the surface of the ground, and thus if a square concrete base with vertical back is given, closely adhering to the original cutting face or soil or rock, this portion may be considered as free from water pressure.

As a general rule, however, the dam base is designed for full water pressure down to its actual base. Below this base a comparatively narrow trench filled with concrete and puddle has often to be excavated to great depths. This is simply a curtain wall to stop any chance of percolation, and is not designed to withstand water pressure, *vide* Fig. 33, Chap. XIII.

(**35**) The formula for the top width of a dam, viz., $\sqrt{H}$, has sometimes to be considerably exceeded owing to other considerations than that of stability, such as providing a roadway along the crest. Further, dams are not of the same depth throughout, the foundations stepping up, and it may be deemed advisable to keep the top width even throughout. In such cases the face slope will have to vary with the reduced base width to avoid waste of material. Were the considerations above mentioned absent, the dam should be constructed with a varying top width, thus ensuring an even slope, in which case the face slope should be designed of one continuous batter, which could be made somewhat steeper than that of the elementary triangular profile.

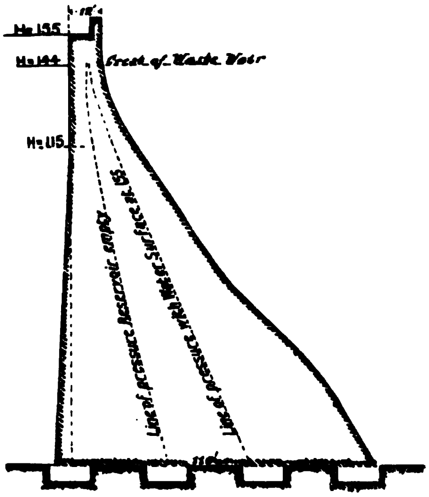

12'
H=155
H=144
Crest of Waste Weir
H=115
Line of pressure Reservoir empty
Line of pressure with Water Surface at 155
116'

Fig. 15.—Peryar Dam.

(**36**) A few examples of dams of considerable size recently constructed will now be given.

Fig. 15 is the section of the Periyar dam, a work recently constructed in the Madras Presidency.

This dam is built entirely of concrete, the maximum admissible pressure being eight tons per square foot. Its greatest height is 155 feet. The line of pressures, reservoir empty, falls some two feet outside the middle third, which could easily have been avoided by a more judicious arrangement of the material.

The object of the bulge in the lower part of the face is not apparent. The profile, Fig. 15, is taken from Buckley's "Irrigation in India."

The dam is 1,300 feet in length and impounds 13,300 millions of cubic feet of water in the valley of the Periyar River, which is fed by the heavy rainfall on the Western Ghauts. By means of this dam the natural outlet of the river is blocked, and the supply is diverted by a tunnel through the water shed of the Peninsula into the comparatively rainless tracts east of the Ghauts. It falls into the Vigar River channel, a stream running towards the east, which forms the source of supply to several canals. Surplus water is passed over a waste weir on the left, and a wide by-wash on the right flank of the dam. The position of the works is shown in Fig. 16, excepting that of the tunnel.

This work was constructed under great difficulties, partly from the unhealthiness and remoteness of the site, and secondly from the Government refusing to sanction the construction of a sluice way through the body of the dam to pass water during construction, which precaution resulted in greatly increasing the cost and difficulty of the work.

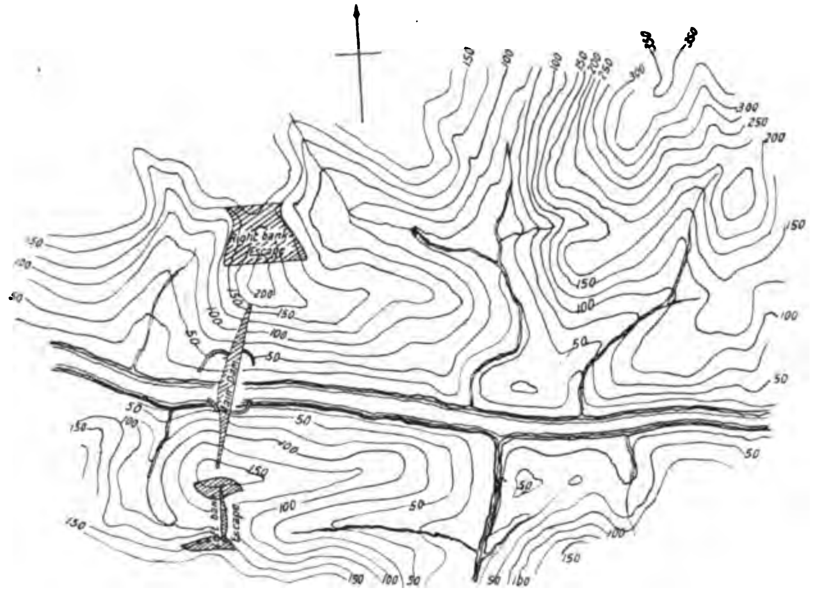


FIG. 16.

Fig. 16 is a general site plan of this great work.

(**37**) The next example, illustrated in Fig. 17, is a section of the Chartrain dam erected in France about fifteen years ago; this has been prepared from one on a larger scale in "Water Works Engineering."

This dam may be considered as the latest exposition of French practice, the Furens dam being one of the first to be constructed on strictly correct principles.

The crest is remarkably light, and is partly carried on arches, which must greatly add to the architectural effect of the elevation. The effective face line of the work is probably situated nearer the outer buttress line than as shown by the hatched portion. The elementary triangular profile has been drawn for purpose of comparison, and the limiting line for low dam calculated and put in. The base width at this point agrees exactly with that of the elementary profile. The design of the lower part is in accordance with the principles already enunciated.

It will be noticed that the back is vertical almost down to the limiting depth; the crest being proportionately so small, it is evident that its effect on the line of pressure, reservoir empty, has been neglected. In the author's opinion the top of the dam is somewhat too light for tropical countries, where extremes of heat and cold in the twenty-four hours are often met with. The height of the dam is 167 feet; it is built of granite with a specific gravity of 2·4, which accounts for its very light appearance. In this, as indeed all dams, the water level is supposed to reach up to actual crest level, as it may do in the occurrence of storms. The section of the Roosevelt Dam in the United States is given in Fig. 17a, Chap. XIII.; in this case the height is 260 feet, and that of the Croton Dam (Fig. 33) 297 feet.

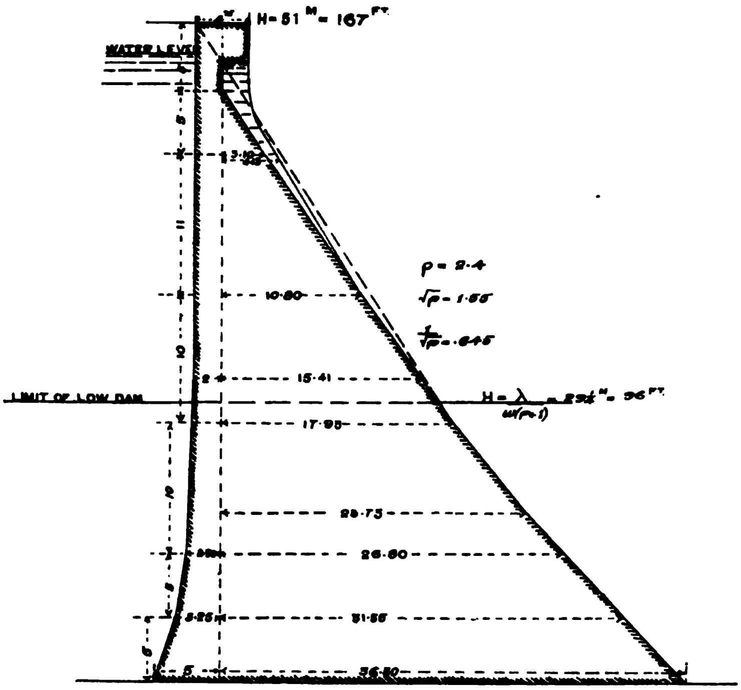


Fig. 17.—Chartrain Dam.

(38) The next two specimens of dams have the peculiarity of being provided with several sluices built in the body of the dam, and are, it is believed, the only instances of this kind on any large scale. The sluices in the Bhatgarh Dam (Fig. 18) are not intended primarily as waste sluices, the work being provided with large waste weirs on either flank, but as scouring sluices in order to remove silt deposit from the reservoir. The Assuan Dam (Fig. 19), which we shall notice later, on the other hand, will pass the whole flood of the Nile through its sluice ways, not being provided with any waste weir or by-wash; the object being to empty its reservoir into the river channel so as to supply Lower Egypt at the period of low Nile, and at the same time to pass all silt-bearing floods down without let or hindrance.

(**39**) Fig. 18 is a section of the Bhatgarh Dam which forms part of the Vira Canal Head works in the Bombay Presidency. Its height is 119 feet, and it has not the appearance of having been correctly designed according to modern lines, the divergence of its face from the hypothenuse of the elementary triangle being too great. It is provided with fifteen low level vents, each 4 feet wide by 8 feet deep, and also turbine sluices.

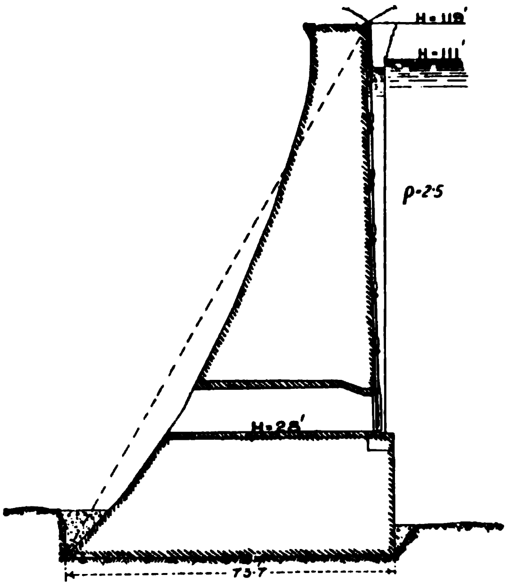


FIG. 18.—Bhatgarh Dam.

This dam is 3,020 feet long and 127 feet high above lowest point in foundations. The prescribed limiting pressure, the vertical component only being considered, is eight tons per square foot, and the average weight of the structure is stipulated not to be less than 160 lbs. per square foot. Although the greater part is composed of concrete, yet this limit was actually exceeded, some of the stone put in weighing as much as 190 lbs. per cubic foot. The concrete was mixed with large blocks of loose stone, which were inserted in the work as it proceeded. These saved mortar and added to the weight and also efficiency of the concrete mass. The sides were lined with ashlar, and the lower portion was built with rubble masonry. The sluice ways were lined with cut granite ashlar blocks. On the section, the elementary triangular profile has been drawn, the base $\frac{H}{\sqrt{\rho}}$ being 71 feet, ρ having a value of over $2\frac{1}{2}$. The actual base is 74 feet, probably to allow for compensation for the vantage.

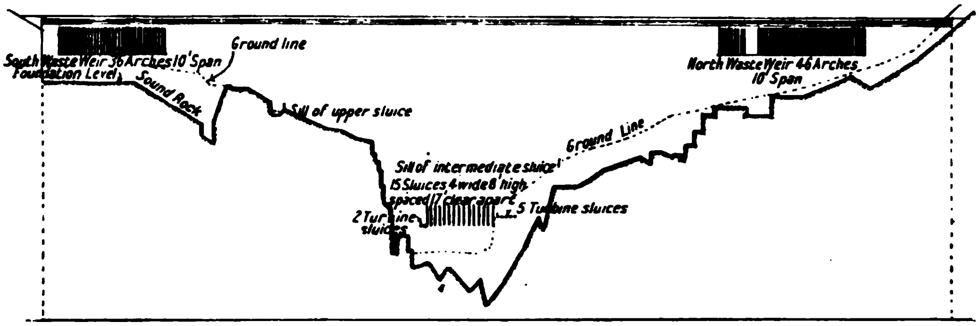

Ground line
South Waste Weir 36 Arches 10′ Span
Foundation Level
Sound Rock
Sill of upper sluice
Ground Line
Sill of intermediate sluice
15 Sluices 4′ wide 8′ high spaced 17′ clear apart
2 Turbine sluices
5 Turbine sluices
North Waste Weir 46 Arches 10′ Span

FIG. 18a.

The fifteen sluices are spaced 17 feet apart, so they do not require any special precautions of limit of pressure with regard to the piers, such as are necessary in the Assuan Dam.

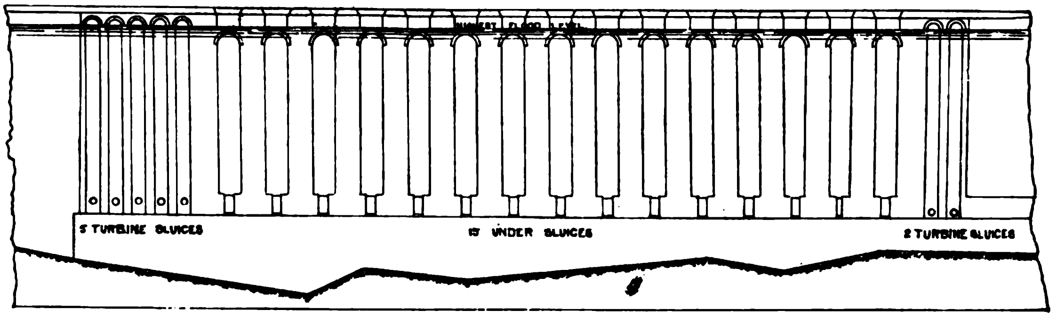


FIG. 18b.

As this work will be again referred to, when on the subject of waste weirs and lifting apparatus, Figs. 18a, 18b and 18c are added.

Fig. 18a is an elevation of the principal part of the dam, showing the position of the sluices.

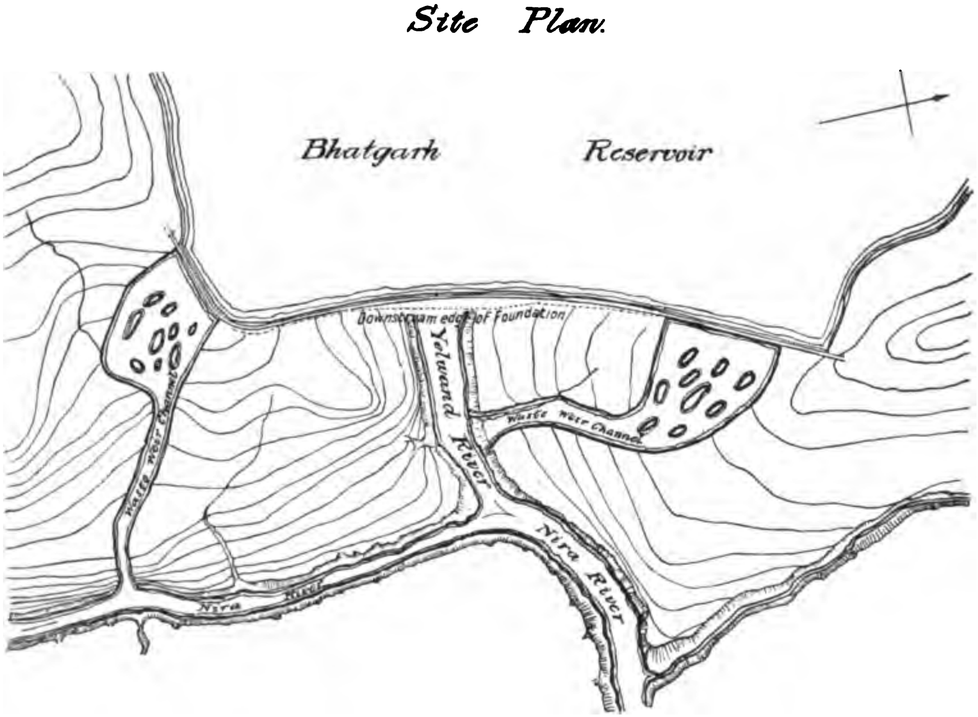


FIG. 18c.

Fig. 18b is a part longitudinal section showing the dam and waste weirs, and Fig. 18c an explanatory sketch plan of the works. These have been prepared from record plans in Buckley's "Irrigation Works in India."

(**40**) The Assuan Dam, as illustrated in Fig. 10, Chap. XIII., is a work lately completed up to R.L. 109·00. This is a remarkable structure, not only on account of its great size and prospective utility, but also from its being of quite a unique type, combining the offices of dam and regulator.

The impounded water in the Nile Reservoir will not overtop the structure in flood time, the dam being provided with sufficient ventage to pass the whole flood discharge of the river. The length of the dam is 1,950 metres, or about 6,400 feet; the greatest head will be 20 metres. The greatest depth now impounded is 28 metres, which means that 8 metres of water will stand on the lower side. The dam is pierced by 140 low-level vents of 2 metres in width and 7 metres in depth, and 40 high-level vents 2 × 3½ metres.

The lower vents are lined with granite ashlar, the upper with cast-iron plates.

The sluice gates are Stoney's patent balance gates working on rollers in grooves with counterbalance weights; they are raised by means of a travelling winch.

The velocity through the under sluices will be 20½ feet per second, and they are capable of discharging 14,000 cubic metres, or 500,000 cubic feet per second.

By inspection of the section it will be seen that a vertical chamber is constructed up stream at each sluice opening, in which the gate works. These chambers extend right up to the roadway. At the ends of the chambers are the grooves in which the rollers work, and the counterweights hang. Each chamber is about 3½ metres long by 1½ metres wide, being arched towards the face on plan. This chamber naturally remains full of water up to the level of the reservoir. The dam is of exceptionally solid construction, as, the whole weight being carried on a reduced length of base owing to the numerous vents, the width has to be extended to bring the pressure within the limit fixed, viz., 4·56 tons per square foot.

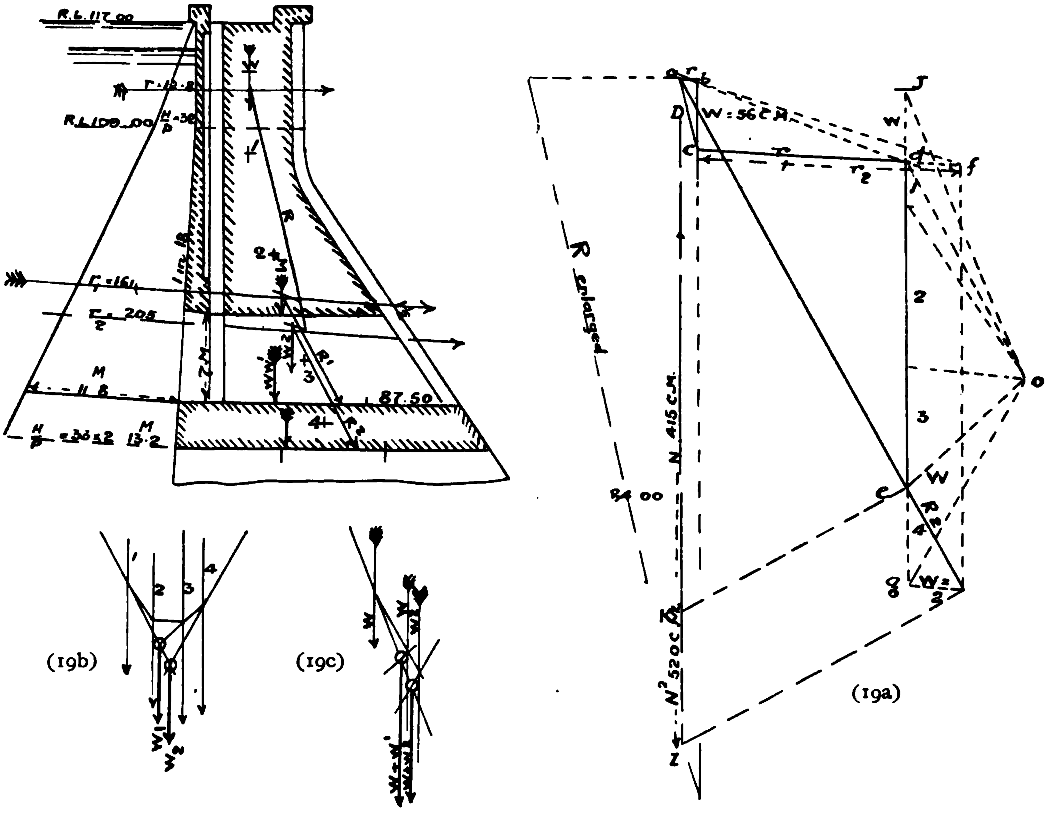
The spaces between the vents (or the piers) are 3 metres wide. After every tenth vent are two abutment piers, each 5 metres wide. At the back of the dam, buttresses 1 metre thick are placed between each vent. These taper from the base, and besides performing duty in adding to the stability of the dam must have a fine architectural effect.

(**41**) It is now intended to raise the dam to its originally devised height of R.L. 117·00, the water level being likewise raised by 8 metres from R.L. 106·00, which it is now, to R.L. 114·00. It will be interesting to investigate the maximum stresses involved at the base of the structure as thus altered.

It is evident that the section of the existing dam was specially designed with a view to its being eventually raised to R.L. 117·00, as otherwise the stipulated limit pressure of 4½ tons per square foot would be far too low a value for economy.

(**42**) Fig. 19 is a presentment of the profile of the dam as it will be when raised to full height. It will be seen that the back is battered 1 in 18 up to R.L. 109, from which level to R.L. 117 it is vertical. The water pressure exerted will be considered as due to a height of R.L. 117·00, *i.e.*, 33 metres above the base of the dam, which is at R.L. 84·00. The extra depth of 3 metres is a necessary allowance for wave action from a long fetch, which would temporarily increase the hydrostatic pressure above what it would be with smooth water in the reservoir.

(**43**) The pressure on the masonry will be graphically ascertained at two levels, one at R.L. 87·50, the base of the vent piers, and the other at R.L. 84·00, the base of the profile. The water level below the weir, which stands at R.L. 86·00, has no appreciable influence in diminishing the effect of that in the reservoir, which is 33 metres above the base of the dam. In the graphical calculations about to be described, the weir is considered divided in three portions: Firstly, that from R.L. 117 to R.L. 109, where the back is vertical—termed *W*; secondly, from 109 to 87·50, the level of the base of the vent piers, *W*1; and thirdly, from 109·00 to 84·00, the base of the profile, *W*2. Again *W*1 is necessarily divided into three areas, 1, 2 and 3, the centres of gravity of which are marked on the profile, and the addition of No. 4 makes up *W*2.



FIGS. 19, 19a, 19b, 19c.—Assuan Dam—Raised Crest.

The superficial area in square metres of the divisions is as follows:—

$W$ = 56 square metres. In $W_1$ and $W_2$ No. 1 = 32.
No. 2 = 126.
No. 3 = 95. Total 253 = $W_1$.
No. 4 = 77. Total 330 = $W_2$.

These areas represent the weight of the masonry, allowance having been made for the buttress and for the vents and sluice gate well.

With regard to the water pressure reduced to weight of masonry, that acting on $W$ is the area of the triangle drawn against the back, having its base $= \frac{H}{\rho}$ or $\frac{8}{2\frac{1}{2}} = 3.2$ metres and its area will be $\frac{H^2}{2\rho}$ or $\frac{64}{5} = 12{\cdot}8$ square metres. This force is termed $r_1$. A value for $\rho$ of $2\frac{1}{2}$ is taken in these calculations, the masonry being of granite of high specific gravity. The water pressure acting on $W_1$, viz., $r_1$, is the trapezium, having a base width of $\frac{H}{\rho}$ or 11·8 and a top width of 3·2, its area being 161 square metres.

That acting on $W_2$, viz., $r_2$, is the trapezium between R.L.'s 84 and 109; its area is 205 square metres. The forces $r_1$ and $r_2$ are inclined parallel to the bases of their trapeziums, *i.e.*, normal to the back face of the dam. They pass through the centres of gravity of their respective areas.

Having obtained the amount of the forces engaged and their direction, the graphical construction can be proceeded with.

In Fig. 19a the line *ab* is first drawn to represent $r$, horizontal in direction and measuring 12·8 metres, the area of the reduced triangle of pressure. From $b$, *bc* is then drawn vertically equal to 56 metres representing $W$. The line *ac*, or $R$, is then the resultant of $r$ and $W$. Again from the point $c$, the line *cd* is drawn parallel to force $r_1$ and in length 161 metres. Then the load-line *de* is set down vertically, containing forces 1, 2 and 3, totalling 253 metres. The line *ae*, or $R_1$, will then be the resultant of $R$, $r_1$ and $W_1$. For if *ad* be joined, then *ad* is the resultant of $R$ and $r_1$ and *ae* is that of *ad* and $W_1$.

For the combination with $W_2$, *eg* is drawn, a continuation of the load-line equal to force 4, and *cd* or $r_1$ continued to $f$, so that $cf = r_2$ or 205 square metres and *fh* drawn, the point $h$ being cut off by *gh* parallel to $r_2$. The line *ah*, or $R_2$ will then represent the resultant of $R$, $r_2$ and $W_2$. This in the diagram appears identical with $R_1$, which is merely a coincidence.

The next step is to construct the ray polygon *dog*, taking any point $o$ as nucleus.

From this the funicular Fig. 19b is framed, in which the centres of gravity of $W_1$ and $W_2$ respectively are found and projected on the profile.

In drawing the line of pressure in Fig. 19, owing to there being two inclinations in the resultant lines of water pressure, viz., $r$ horizontal and $r_1$ and $r_2$ inclined, Haessler's system has been employed. The first line $R$, reciprocal to $R$ in Fig. 19a, starts from the intersection of $r$ with $W$ and from the point at which it cuts $r_1$, a line is drawn backwards (parallel to *da* in Fig. 19a) intersecting the vertical force $W_1$. From this point $R_1$ runs to the base of $W_1$, parallel with and reciprocal to $R_1$ in Fig. 19a.

For combination with $W_2$ the line $R$ is continued through to cut $r_2$, and from this point the return line is parallel to *fa* in Fig. 19a, and $R_2$ starts from this latter intersection to the base of $W_2$, viz., R.L. 84·00.

We have thus obtained the incidence of the resultant pressures reservoir full, viz., $R_1$ and $R_2$ on the base lines at R.L.'s 87·50 and 84·00. It still remains to obtain the corresponding resultants when the reservoir is empty. These are clearly the resultants of the vertical forces $W$ with $W_1$ and $W$ with $W_2$. To effect this first on Fig. 19a *dj* is set off equal to *bc* or $W$ and the ray *jO* joined.

Then in Fig. 19c a reciprocal chord polygon is constructed across the projection of the lines of centre of gravity of $W$, $W_1$ and $W_2$. The first chord cutting $W$ is parallel to ray *jO*, the second cutting $W_1$ and also $W$ parallel to *dO*. The chord from the first of these intersections parallel to *oe* cuts the original line and gives the centre of gravity of $W$ and $W_1$. Another third chord from the second intersection, viz., that of $W_2$ parallel to the ray *og* again cuts the first chord prolonged, and the point thus obtained is the centre of gravity of $W$ and $W_2$.

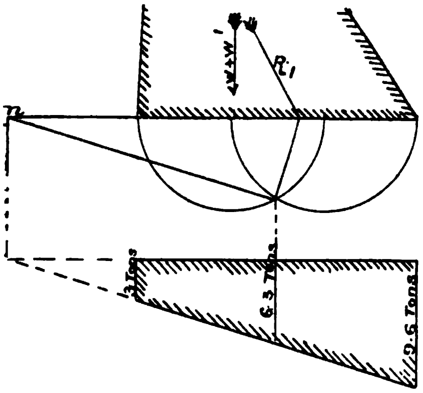


FIG. 19d.

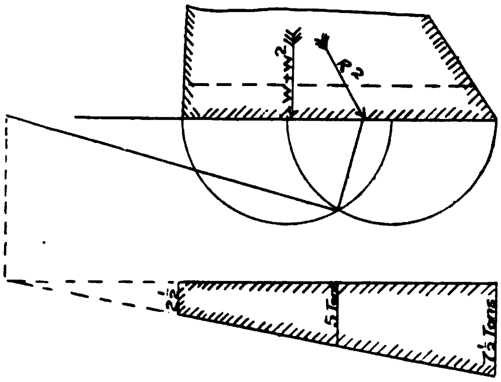


FIG. 19e.

These two vertical resultants are now projected on to the profile intersecting their respective bases.

We now see that on both bases viz., at R.L.'s 87·50 and 84·00, the vertical resultants (reservoir empty) fall just within the middle third of their base, while the inclined resultants $R_1$ and $R_2$ fall well within the middle third of their bases.

The next point to be determined is the maximum intensity of pressure due to $R_1$ and to $R_2$. As we have already seen, the amount of this is not the value of $R$ but of $R$ sec. $\alpha$ or $N_1$.

In Fig. 19a the normals *ek*, *hl*, are drawn intersecting the vertical through *a* in *k* and *l*, then *ak*, termed $N_1$, represents the intensity of pressure due to $R_1$, and *al* ($N_2$) that due to $R_2$, and their lengths 415 and 520 respectively measure the pressure in cubic metres.

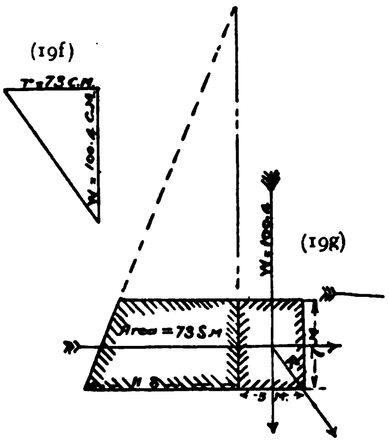
In Fig. 19d the base of the dam at R.L. 87·50 is reproduced with the incidences of $R_1$ and $(W+W_1)$. The latter, being the less, need not be considered. With regard to $R_1$, or rather $N_1$, its value, as we have seen, is the weight of 415 cubic metres of masonry at specific gravity $2\frac{1}{2}$. As a cubic metre of water weighs 1 ton, one of masonry will weigh $2\frac{1}{2}$ tons. Hence the distributed pressure on the base is $415 \times 2\frac{1}{2} = 1{,}037\frac{1}{2}$ tons.

Divide this by length of base or $21\frac{1}{2}$ metres, and the quotient is 48·2 tons the average pressure per metre run. To convert metres into square feet, divide again by 10·76 and $\frac{48{\cdot}2}{10{\cdot}76} = 4{\cdot}48$ tons per square foot average pressure. As, however, the vents take up two-fifths of the space lengthwise, the pressure on the piers will be seven-fifths greater, or $= \frac{4{\cdot}48 \times 7}{5}$ $= 6{\cdot}3$ tons per square foot average pressure.

To obtain the maximum pressure, the procedure previously explained is shown in Fig. 19d, from which the maximum pressure is deduced as 9·6 tons per square foot on the base of the piers—at R.L. 87·50.

In the same way the maximum pressure on the base of the solid dam at R.L. 84·00 is shown to be $7\frac{1}{2}$ tons (*vide* Fig. 19e).

From the above we see that the maximum pressure on any part of the masonry is that at the base of the piers, and that it is under 10 tons per square foot, which is probably the ruling limiting stress. Hence the structure when raised is perfectly secure under the increased head of 8 metres it will have to uphold, and in fact was originally designed for this purpose, though nothing was said about it at the time, for good reasons.



FIGS. 19f, 19g.

(**44**) We will now consider one more point, and that is the lateral pressure induced on a pier supposing its neighbouring vent is closed.

In Fig. 19f the cross section of the pier is shown; it is 5 metres wide and 7 metres high.

The pressure against it is represented in cubic metres by the area of the trapezium, viz., 73 square metres. This pressure is horizontal, acting through the centre of gravity of the trapezium of pressure.

The component vertical weight consists of $(W + W_1)$ or $56 + 253 = 309$ cubic metres. This weight is that on a section 1 metre wide, hence it must be multiplied by the width of the piers, viz., 5 metres, and divided by the length. $W$ will therefore be $\frac{309 \times 5}{21{\cdot}5} = 72$ cubic metres nearly. This, however, has again to be multiplied by $\frac{7}{5}$ to allow for the vents (the load of 309 cubic metres being considered distributed evenly along the length of the dam); $W$ will therefore become $7{\cdot}2 \times \frac{7}{5} = 100{\cdot}4$. The triangle of forces is shown in

Fig. 19g, and the reciprocal of the resultant line drawn on the wall section in Fig. 19f cuts the base just within the outer edge.

The lateral pressure on piers of regulators, undersluices, and similar works is a matter always requiring consideration, as the neglect of it may result in disaster, as was notably the case in a large work in India. When one bay of a regulator is closed and the others remain open, a contingency that may occur at any time, lateral stress is induced in the pier, the thickness of which must be such that the resultant line of this lateral pressure, combined with the weight of the pier, must pass within its base. This should be irrespective of the fact that a pier acts as a girder supported at both ends by the arch above and the friction on the base at the other.

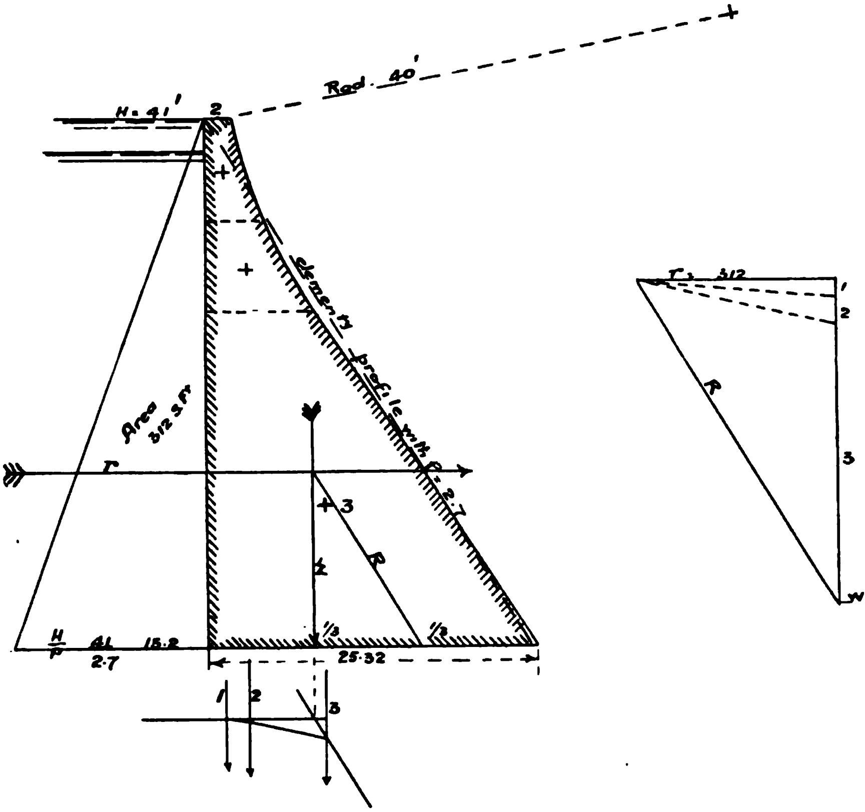


FIG. 20.

(45) The profile of the Burraga Dam, lately constructed in New South Wales, is taken from Vol. CIII., " Min. Pro. Inst. C.E."

Inspection of the section, Fig. 20, shows that the profile corresponds more or less closely with the elementary triangular profile, which is dotted in. The back of the dam is vertical. As the weight of the material employed is 170 lbs. per cubic foot, the value of $\rho$ will be 2·7, and this has been adopted in the graphical calculations. The peculiarity of the section, otherwise almost unexceptionable, is the extremely narrow top width adopted. This, though theoretically correct, is under serious practical objections. The work, being situated in a sub-tropical climate, is subject to extreme alternations of heat and cold, and the ensuing alternating expansion and contraction of the material of the dam induces severe undeterminable stresses which may cause rupture at or near the crest which is the most exposed to such influences, and from its want of width least able to withstand them. Under such circumstances a solid wide crest is a necessity.

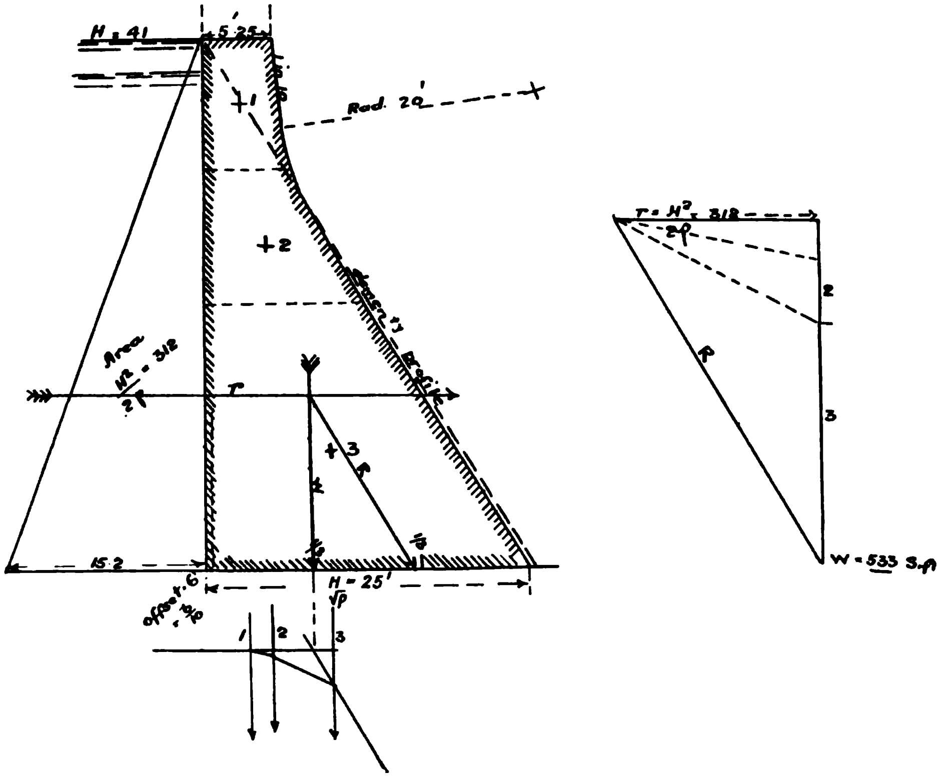

H = 41
5·25
+1
Rad. 20′
+2
Area $\frac{H^2}{2\rho}$ = 312
T
+3
R
15·2
offset ·6
$H = 25'$ / $\sqrt{\rho}$
1
2
3
$T = \frac{H^2}{2\rho} = 312$
2
3
R
W = 533 S. ft

FIG. 21.

(46) In Fig. 21 a revised profile is given in accordance with the rules already enunciated. In these the top width is made $\frac{1}{4}\left(\frac{H}{\sqrt{\rho}} - \frac{H}{10}\right)$ *i.e.* $5\frac{1}{4}$ feet. A back batter is introduced and the base width made $\frac{H}{\sqrt{\rho}}$. The water pressure in both instances is calculated from the crest of the dam. The excess of sectional area in Fig. 21 over Fig. 20 is 30 square feet out of a total of over 533 square feet—*i.e.*, 6 to 7 per cent.

The profile in Fig. 21 has not such a good appearance as Fig. 20, but it is a much sounder design. In Fig. 20 the vertical component *W* falls outside the middle third, demonstrating the necessity of a small back batter to the section. The fore slope in Fig. 21 is made 1 to $1\frac{1}{2}$. This is only suitable where the value of $\rho$ is $2\frac{1}{4}$; but in this case $\rho$ is 2·7, consequently this batter should be steeper. Sections of the Roosevelt and the New Croton dams are given in Chap. XIII., Figs. 17a and 33c.

## II. Arched Dams.

**(47)** The subject of dams cannot be considered as completed, without reference to those which are arched on plan, of which some notable examples have been constructed in the United States.

An arched dam differs from the ordinary type of gravity dam in that the thrust of the water is conveyed to the rock abutments on either side on the principle of an arch; the problem is thus not identical with that of a gravity dam in which the thrust of the water, compounded with the weight of the masonry, is transmitted to the base of the structure.

In an ordinary bridge arch with horizontal spring line, and with spandrel filling up to a horizontal line above the crown, the line of pressure in the arch ring forms a parabolic curve. In the case of an arch ring of equal depth throughout, with no spandrel filling, the curve of pressure is a catenary. In the present instance the arch is vertical and the pressure exerted by the water on the extrados is everywhere radial in direction, not as in the two former cases, at right angles to the spring line; consequently the curve of the line of pressure must be circular. The thrust also is the same right through, not increasing with the declination of the arch curve towards the haunches. The value of this thrust at every point on the curve is $RHw$, $R$ being the radius of the extrados, $H$ the depth of water at this point, and $w$ the unit weight of water, or $\frac{1}{36}$ ton per cubic foot.

The water pressure is *nil* at the surface and increases as the square of the depth, hence the theoretical section of the arch will be a triangle, but in practice a trapezium, owing to the necessary width given to the crest.

**(48)** Let $\lambda$ be the limiting stress it is proposed to adopt, which is very much higher than that admissible in a gravity dam, then $b$, the required width of the arch, at any depth $H$, is obtained by the following formula :—

$$b = R\left(1 - \sqrt{1 - 2\frac{Hw}{\lambda}}\right)^{*} \qquad (1)$$

Transposing this equation—

$$S \text{ or } \lambda = \frac{2\,Hw}{\frac{b}{R}\left(2 - \frac{b}{R}\right)} \qquad (2)$$

The last formula decomposed, is $RHw \times \frac{2\,R}{R + r}$, ($r$ = radius of the intrados). This fraction multiplicand increases induced stress to maximum stress in the section.

**(49)** Fig. 22 is the section of the recently constructed Shoshone Dam. The profile, with the resultant lines of stress worked out, is, as will be seen, trapezoidal in form down to $A$, the bed level of the river, *i.e.*, to R.L. 62·00. Below this level the sides are vertical. In adopting this form the designers seem to have been under the very erroneous impression that the hydrostatic pressure does not continue down to the rock base at *B*, or else that this lower pressure from *A* to *B* was neutralised by the back pressure of the water and gravel in the bed of the Shoshone.

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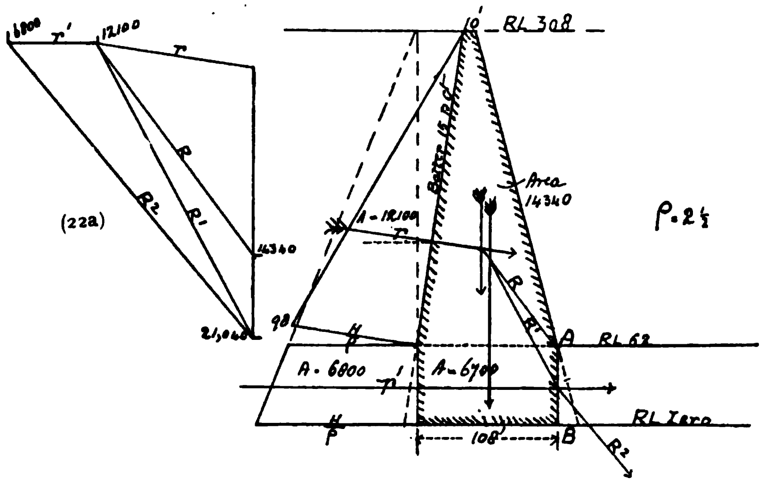
* "The Principles of Water Works Engineering."

As hydrostatic pressure must be considered to exist down to the solid foundation, and further, the presence of water from *A* to *B*, down stream of the dam, has a quite inappreciable effect, it is evident that the base width below *A* ought to continue to increase in the same proportion, as above that point, giving a final base width, as shown by the dotted lines on Fig. 22, of about 135 feet, instead of 108 feet as designed. If this were done the final resultant $R_2$ would not fall outside the toe, as is now the case.



FIGS. 22, 22a.—Shoshone Dam.

**(50)** The maximum stress induced in the arch ring at the base is obtained from formula (2) in which $H = 308$; $R$ (the radius at the extrados) is 200 feet nearly, and $b$ is 108; then $\frac{b}{R} = \cdot 54$ feet and

$$S = \frac{2 \times 308 \times \frac{1}{36}}{\cdot 54\ (2 - \cdot 54)} = \frac{17 \cdot 11}{\cdot 788} = 21 \cdot 7 \text{ tons.}$$

This unit stress is moderate when compared with that in the Bear Valley Dam, the section of which is given in Fig. 24. This section has a top width of 2½ and a base width, at a depth of 48 feet, of only 8½ feet, the external radius being also much greater than in the case of the Shoshone Dam, viz., 335 feet. The value of *S* in this case works out to the astonishing figure of 53 tons per square foot.

In the Sweet Water arched dam, another American work,

$$H = 90 \text{ feet}$$

$$R = 222 \text{ feet and } b = 46 \text{ feet. } \frac{b}{R} \text{ is then } \frac{46}{222} = \cdot 207$$

$$\text{Then } S = \frac{2 \times 90 \times \frac{1}{36}}{\cdot 207\ (2 - \cdot 207)} = \frac{5}{\cdot 371} = 13 \cdot 48 \text{ tons.}$$

**(51)** It is deemed suitable that the determination of the base width of an arched dam should be guided by the position of the incidence of the resultant line of pressure within base, supposing the section to be considered as a gravity dam; the maximum unit pressure when exceeding 10 tons being met by steel bar reinforcement.

This has apparently received attention in the design of the Shoshone Dam section.

In Figs. 22 and 22a the incidence of the resultant, as in a gravity dam, is shown worked out by graphical process at two depths, viz., *A* and *B*. At *B* the final resultant $R_2$ falls outside the toe of the base. If the face and rear batters were continued to the actual base, as has already been suggested, the incidence would be just at the toe. At the point *A* it falls well within the base.

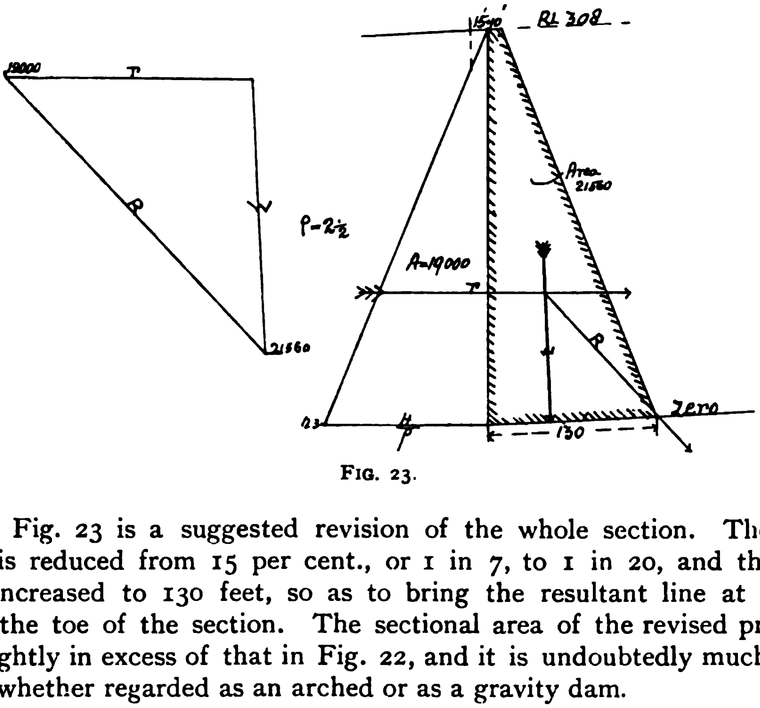


FIG. 23.

**(52)** Fig. 23 is a suggested revision of the whole section. The back batter is reduced from 15 per cent., or 1 in 7, to 1 in 20, and the base width increased to 130 feet, so as to bring the resultant line at or just within the toe of the section. The sectional area of the revised profile is only slightly in excess of that in Fig. 22, and it is undoubtedly much more stable, whether regarded as an arched or as a gravity dam.

In this case the maximum stress, or *S*, is less than in Fig. 22. Here $R = 175$, being $(150 + 10 + 15)$, $b = 130$

$$\frac{b}{R} = \frac{130}{175} = \frac{26}{35} \therefore S = 2 \times 308 \times \frac{1}{36} \div \frac{26}{35}\left(2 - \frac{26}{35}\right)$$
$$= \frac{188650}{10296} = 18{\cdot}3 \text{ tons.}$$

**(53)** There seems no reason why an arched dam should be designed nearly equilateral in section, as is the case in Fig. 22. The equilateral form certainly brings the centre of gravity of the dam (reservoir empty) near to the centre of the base, thus reducing the maximum intensity stress due to the vertical load. On the other hand, the effect of the rear batter, with reservoir full, is to load the final base, as well as the arch, with a considerable weight of water, which is detrimental from the standpoint of maximum unit pressure. A small batter is certainly useful for constructional reasons, from the face bond it automatically provides.

**(54)** The section in Fig. 24 of the Bear Valley Dam must be considered as quite exceptional, but is most useful as an example of what enormous stresses the arched form of dam is capable of sustaining, even without reinforcement of steel. As remarked by an eminent authority, any sudden shock would probably cause its immediate collapse.

The section in Fig. 24 would be better if reversed, *i.e.*, with the vertical side at the back instead of at the face.

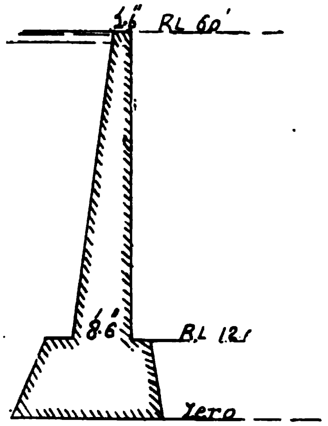


Fig. 24.—Bear Valley Dam.

**(55)** The Shoshone Dam will be composed of cement concrete fortified with steel rails, where such reinforcement is necessary, *i.e.*, in the lower portion of the dam, where the compressive stresses exceed what is deemed the safe limit. An arched profile on plan is the best form for any gravity dam as it permits of variation in length due to expansion and contraction which a straight dam does not, and the stresses thus produced are often extreme.

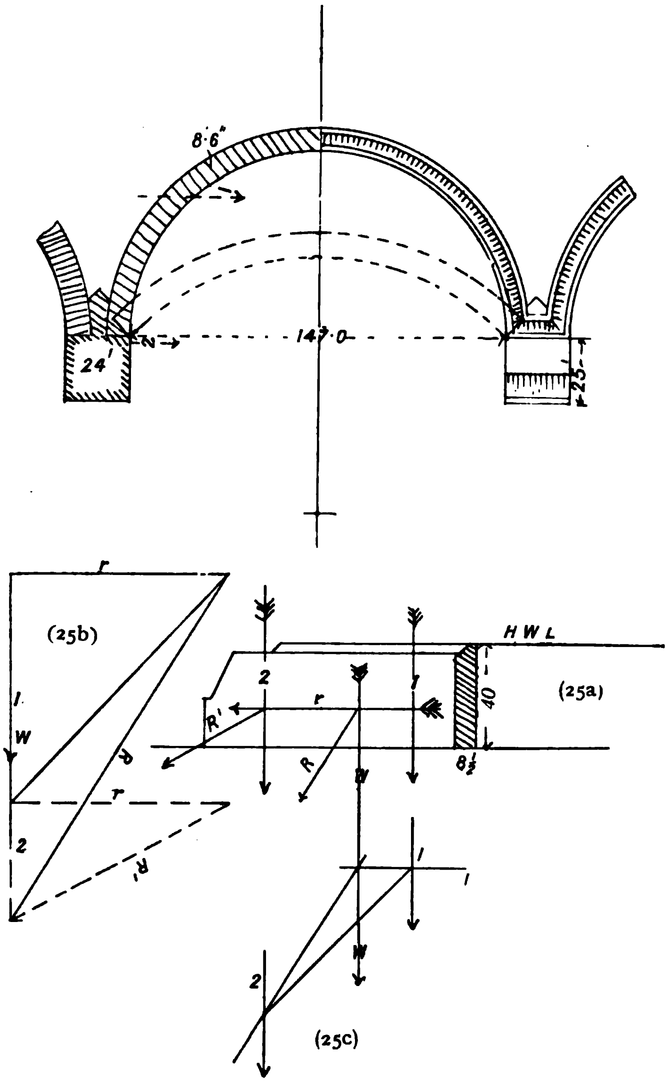
## III. Panel and Buttress Dams and Weirs.

**(56)** Another form of arched dam, and also weir, is that of the buttress and panel type, similar in style to the retaining walls in the old Metropolitan Railway stations in London.

One notable example is the Mir Alam tank dam at Hyderabad Deccan. This structure is curved on plan as a whole, and besides, the dam is formed by semicircular vertical arches abutting on pier buttresses. The spans, for some reason, are not all alike, varying from 70 to nearly 150 feet diameter. Figs. 25 and 25a are sectional plan and transverse section of the largest panel.

It is evident that the semicircular form of the panel arch is adopted, in order to avoid any lateral thrust on the buttress piers, as the component of the water pressure in a direction parallel to the axis of the dam is clearly balanced.

The adoption of segmental arches with a moderate rise of, say, one-quarter the span, would result in considerable economy of material, but the horizontal thrust at the springing would be unbalanced, and is conveyed to the next arches and eventually to the abutments on each side, so that in case of possible failure of the buttress piers the work might be wrecked. This danger could, however, be avoided by occasional abutment piers or by tying some of the arches with steel tie rods. After all, it is no more than what any arched bridge or aqueduct is liable to. The further principles involved will be treated after the exhibition of other examples.



FIGS. 25, 25a, 25b, 25c.—Mir Alam Arched Panel Dam.

(57) Two similar works are mentioned in "Irrigation Engineering" (Wilson), viz., the Belubula Dam in New South Wales of the following dimensions: Length 430 feet, depth 60 feet. There are six buttress piers at 28 feet centres, and from 5 feet to 12 feet thick and 40 feet in length. The brick panel arches are inclined 60° to the horizontal and are 1½ feet thick at crest and 4 feet at base.

The latest type is the Ogden Dam, in Utah, U.S.A., built of concrete, height 100 feet, length 400 feet, the buttress piers are 16 feet thick, the spans of the panel arches being 32 feet in the clear, the arches are vertical, from 6 feet to 8 feet thick. These arches are covered with steel plates to render them watertight. The saving in adapting this type over the ordinary gravity dam is stated to amount to 15 per cent. A good design should save quite 25 per cent.

(58) Fig. 26 is a representation of the Theresa reinforced concrete weir recently constructed in New York State. This work is an overflow weir, not a dam, but will be treated in this chapter, as it is of the flat panel buttress

type. It is of small dimensions, being 11 feet in height only. The design consists of a deck inclined 30° composed of 6 inches of concrete, reinforced by steel bars ¾ to ⅞ inch diameter, spaced 6 inches apart and further by expanded metal; this deck is carried by buttress piers 1 foot thick, spaced at 6 feet centres.

The design cannot be commended in any way. A weir of this height, of solid construction, with tension not barred, would have a sectional area of about 50 square feet per foot run. The section in Fig. 26 is estimated at 33⅓ square feet per foot run. The saving in cost of concrete would be more than made up by that of the extensive steel reinforcement necessary in the inclined deck.

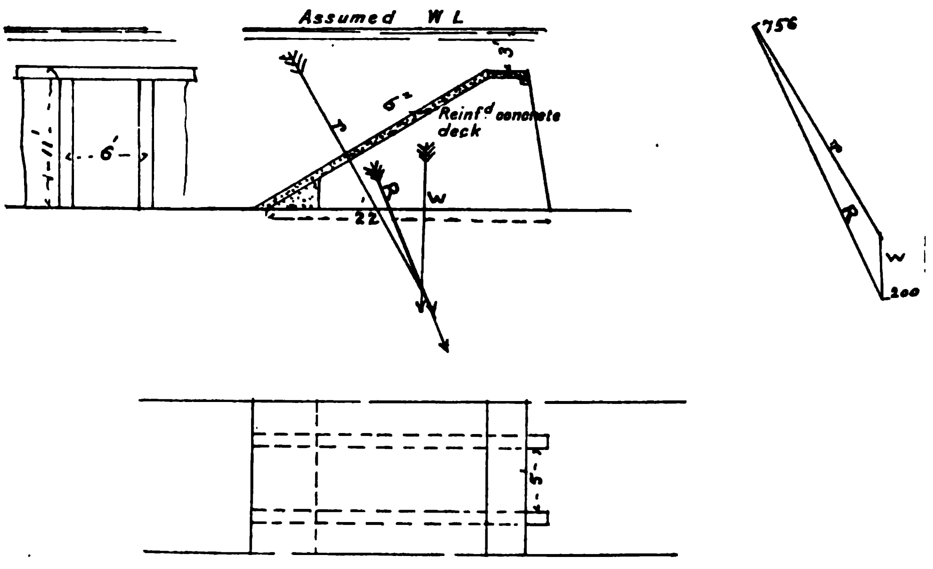


FIG. 26.—Theresa Panel Weir.

(59) To the critical eye it is at once evident that the structure would be improved in the point of suitability, prime cost and statical stability by simply reversing it, and substituting vertical arched panels for the inclined deck. This has been done in Fig. 27.

By this simple means the useless weight of water lying on the deck, which unnecessarily increases the pressure on the buttress, is got rid of or very nearly so. This accomplished, we are enabled to increase the spans to 6 feet, from the 5 feet originally adopted and to make the brick arched panelling 9 inches thick against the 6 inches of the flat deck. With these alterations the comparative area per foot run of Figs. 26 and 27 is as follows:—Fig. 26, Concrete 33⅓ cubic feet. Fig. 27, 32½ cubic feet; in addition to which the steel reinforcement in Fig. 26 is absent in Fig. 27. As regards the pressure on foundations, that at buttress base in Fig. 26 is estimated 3½ tons, and that in Fig. 27 at 2½ tons per square foot. Wider spans of 10 feet, if adopted, would still further reduce the cost.

**(60)** In designing a dam of this description, the following points require consideration:—

First, the most suitable span and versed sine to be adopted for the arch, for from these data the radius is obtained, by which the thrust in the arch ring and the requisite thickness is found by using formula (1), viz.:—

$$b = R(1 - \sqrt{1 - \frac{2Hw}{\lambda}}$$

in which $\lambda$ is the limiting compressive stress considered suitable, $R$ the radius of the intrados, $H$ any depth, and $w$ $\frac{1}{36}$ ton.

With regard to the dimensions of the buttress, the thickness should be some proportion of the span, say not less than ·2 $S$ up to 100 feet span and less beyond, whereas the length will have to be tentatively assumed and the result tested graphically until a suitable dimension is arrived at. $S$ should $= H$.

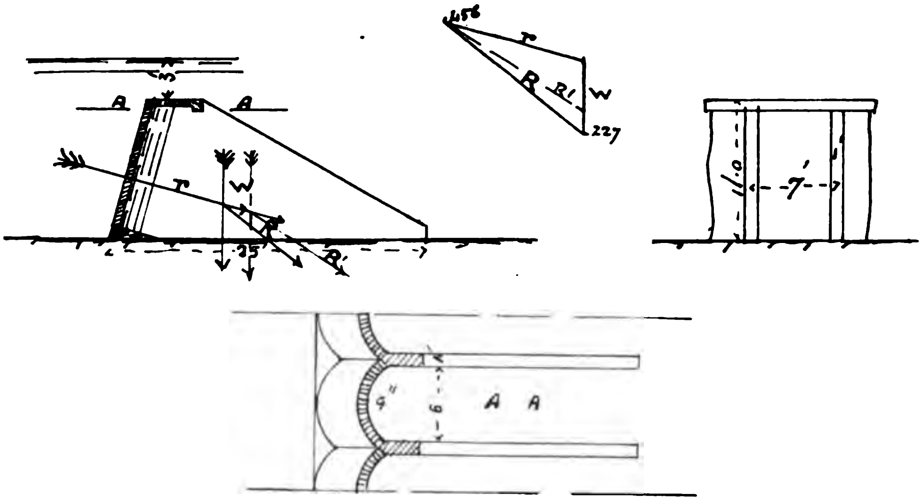


FIG. 27.

It is self-evident that the horizontal pressure of the water in the reservoir acting on the buttress must be identical, whatever disposition is made of the arch on plan. Thus in Fig. 25 the outward thrust on the buttress piers is the same whether the arches which transmit the principal part of this thrust be semicircular or segmental (as shown in the dotted line), or whether a flat panel were made use of as in Fig. 26, and this thrust is $\frac{H^2 w l}{2}$, $l$ being the space apart of the centres of the buttress piers. If the arch is not vertical, but inclined, the thrust will also be inclined in direction, and will be $\frac{H_1 H w l}{2}$, $H_1$ being the inclined height of arch. This applies strictly to a dam, not a weir.

The action of this thrust on the buttress is two-fold, viz., firstly, an overturning moment; and secondly, a shearing thrust.

In the first case it is evident that the weight of the arch panels cannot be dissociated from that of the buttress, as the whole forms one structure, and must overturn together. This has been taken into account already in the graphical calculations outlined in Figs. 26 and 27. If correct in one case with a flat panel, the theory must apply in equal force in that of Fig. 27, where the panel is arched in plan.

The length of the base of the buttress must be such, that the incidence of the resultant line of pressure be within the outer third length of this base, and, further, the prescribed limiting stress on the masonry must not be exceeded. This base should be measured from the extrados of the arch at crown.

The second action of the thrust is to cause failure by the sliding of the pier base on its foundation, or on any higher plane. In this case the weight of the arch through which the thrust is conveyed should be neglected as an asset, and the buttress unaided must be equal to the task of resisting this shearing action. In Figs. 25a, 25b, and 25c the vertical forces 1 and 2 represent the weights of the panel and of the buttress respectively, while the horizontal line *r* represents the thrust of the water on the pier. This latter amounts (reduced to a masonry base) to $\frac{H^2 \times l}{2p} = \frac{1600 \times 171}{2 \times 2} = 68{,}400$, *w* being as usual omitted as a common factor.

The forces 1 and 2 are the cubical contents of the arch panel and of the pier respectively, viz.: (1) The two half-arches up to the point *a* in Fig. 25 have a cubical content of 71,360 cubic feet. (2) The buttress, including the portion of the arches near the spring line within the point *a*, measures 38,400 cubic feet.

The centre of gravity of these two forces on Fig. 25a is *W*, obtained by means of the funicular, Fig. 25c.

The position of the incidence of *R*, reciprocal to *R* in the force polygon, Fig. 25b, is actually within the rear of the buttress pier. Thus we see that the structure is perfectly safe from being overturned. If a segmental arch were adopted instead of the semicircular, the position of *W* and of *R* would of course be much more forward, *R* falling within the base of the pier.

Now with regard to the second consideration. The resultant line of pressure will be composed of *r* and (2) represented in Figs. 25b and 25a by the line $R_1$. One condition is that the incidence of $R_1$ must be within the toe of the buttress, and another is that its inclination with the horizontal must not exceed 1 horizontal to 1 vertical. If the direction of $R_1$ is flatter than 1 to 1, the weight on the pier must be increased, which can be effected by augmenting the inclination of the arched panel to the vertical. When the inclination of the resultant pressure is at about 3 horizontal to 4 vertical, there is no tension on the cementing material of the piers, the thrust being absorbed by the friction of surfaces alone. When this slope is exceeded tension will be developed in the horizontal joints of the masonry or in the concrete; to avoid which the masonry or brickwork in a buttress should be built with inclined courses, which will effectively dispose of sliding action. The incidence of $R_1$ is also shown in Fig. 27. In Fig. 25 the buttress pier should have been designed of greater length.

With segmental panel arches, the thrust is resolved at the springing into two components at right angles to each other, one down the buttress and the other parallel to the axis of the dam ; this latter is balanced by a similar thrust from the adjoining half-arch, but the last arch of the series conveys it to the abutments. This action is identical with that produced on a bridge with horizontally sprung arches.

As this type of dam is undoubtedly economical, and lends itself to the employment of reinforced concrete, it is probably destined to more extensive adoption in future even where solid rock is not met with in the foundations.

**(61)** A further example of the flat panel and buttress type of weir or dam is given in Fig. 28 of the Schuylerville weir in the United States, the plans of which have been prepared from Fig. 535, of " Reinforced Concrete " (Dunn and Marsh). Inspection of these figures will show that the profile is a development of that in Fig. 26 in this respect, that the reinforced flat concrete panel is carried over the crest and down the further side of the buttresses, forming a large box, with the buttresses as partitions. These are pierced by two sets of doorways, so that communication can take place at two levels, the upper one being provided with a planked pathway. The buttresses, which are at 8 feet 1 inch centres, are further supported laterally by three reinforced concrete beams, which run right through the whole length of the weir. This weir is subjected to a head upstream of 31 feet of water, and the inclined deck has consequently to be heavily reinforced with corrugated steel bars to enable it to support the load of water it carries.

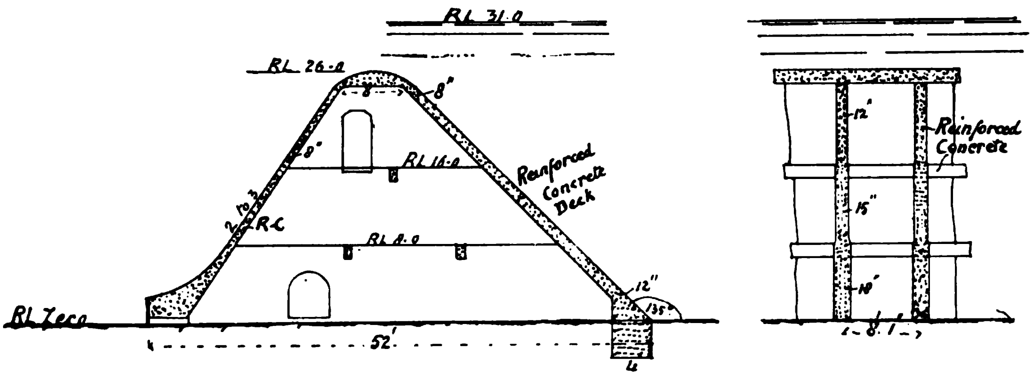


FIG. 28.—Schuylerville Box Panel Weir.

As in the case of the Theresa weir, this section would be improved if reversed, but a still better plan would be to abolish it altogether for the following reasons :—

(1) The deck, if constructed of arched instead of flat panels, would not require any reinforcement whatever, nor increase of section. The concrete arches would then be wholly in compression, *i.e.*, in the most favourable situation for withstanding stress, and as arched dams can safely bear a stress of quite 20 tons per square foot, the section could be reduced in thickness or the spans increased.

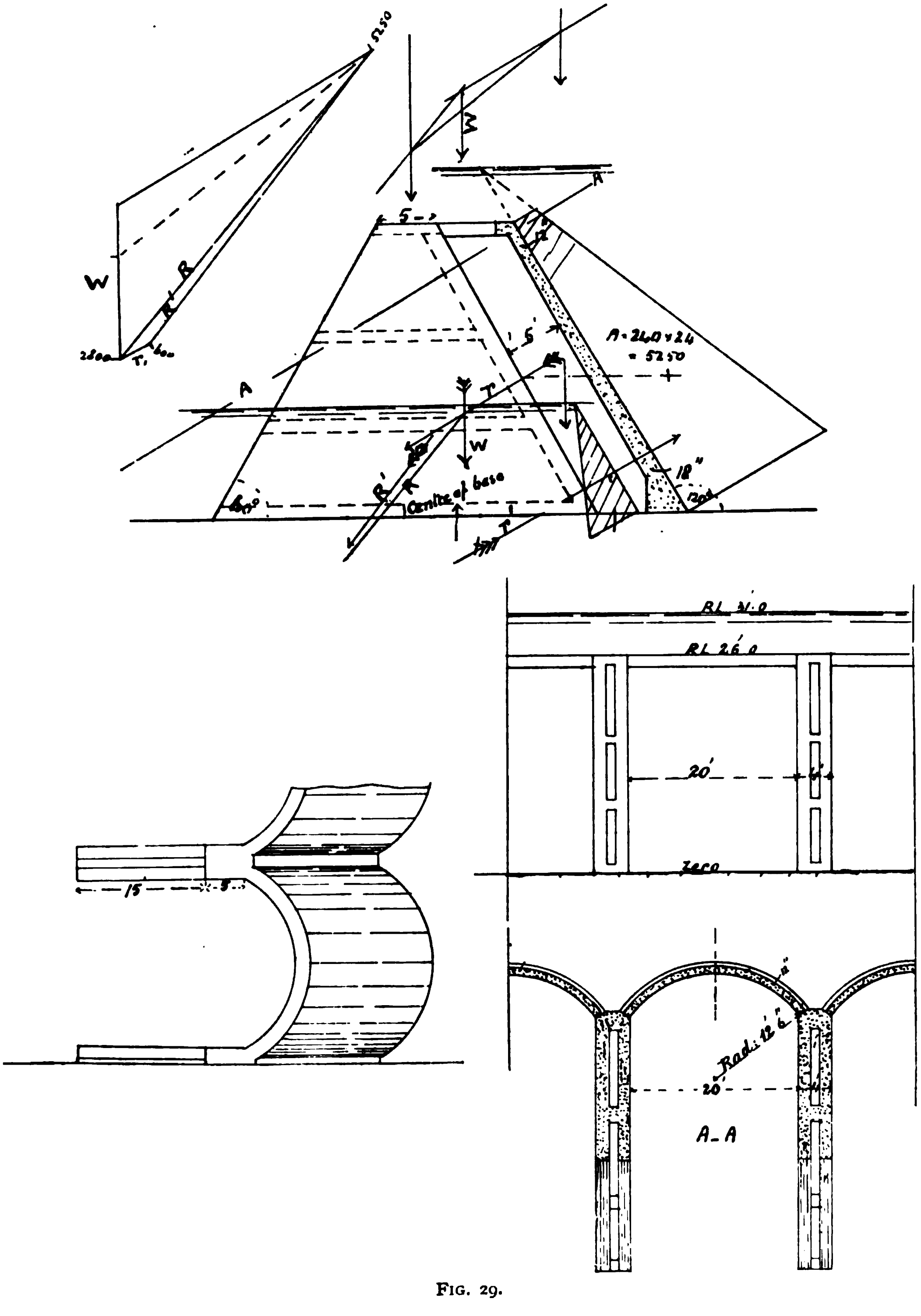

5250
W
R' R
2500
T₁
600
W
A
5
5'
A = 240 × 24
= 5250
T
W
R'
R
Centre of base
60°
18"
120°
T'
RL 31.0
RL 26.0
20'
4'
zero
15'
5'
6"
Rad: 12' 6"
20'
A–A

FIG. 29.

(2) The down-stream panel is quite useless for structural reasons; a direct overfall, as in Theresa weir, is deemed far preferable for the destruction of velocity, than a sloping face, which tends to encourage it.

(3) With this considerable head of water, the buttresses are too close together for economy, and advantage could be taken of the arched profile of the panels, to considerably increase the distance apart of the buttresses, which cannot be effected with the flat deck.

**(62)** An alternative design has been prepared on these lines for the same work in Fig. 29.

In this the profile is equiangular, the upright arches being inclined at 60° with the vertical, as are also the lower ends of the buttresses. Twenty feet spans have been adopted with panel arches of an average thickness of 15 inches. Twenty-five feet spans, *i.e.*, $S = H$, would give even better results.

The buttresses are hollow, formed of two parallel 18-inch concrete walls, connected together in four places by horizontal longitudinal beams of concrete. They are thus given considerable lateral stability with a very small increase of sectional area.

**(63)** The maximum unit pressure on the lowest lamina of the panel arches is by formula 2, par. 47, 8·2 tons per square foot. In this formula $H$ is 31 feet. The normal unit pressure on the extrados of the arch, if inclined ever so much, never exceeds that due to the vertical depth of water. On the other hand the total pressure is greater, as the inclined length of the arch exceeds the vertical height. For this reason the normal unit stress in the arch ring due to the water pressure is identical, whatever be the position of the arch relative to a vertical line. At the same time the weight of the arch when inclined, induces thrust in the arch ring in the same manner as in a horizontal arch, besides a compressive stress down the axis.

**(64)** In the diagram Fig. 30, the vertical $W$ represents the weight of the arch. The two components of this force, resolved in the inclined direction of the arch axis, and in one normal to it, are the forces $p$ and $n$. As the angle between $p$ and $W$ is 30°, $n = W \sin 30° = ·5\ W$, and $p = W \cos 30° = ·866\ W$. Thus half the weight of each lamina of the arch 1 foot deep is conveyed through it to the abutments. The thrust thus induced at the crown by the normal force $n$ will be $Wrb \times ·5$, and the unit stress $wpr \times ·5$, or $·5 \times 12·5 \div 16 = ·4$ ton, $r$ being the radius of the intrados (*vide* par. 13, Chap. IV.). The maximum stress in the cross section will then be (*vide* par 48) $·4 \times \frac{2R}{R + r} = ·4 \times \frac{28}{26·5} = ·42$ tons. This stress, unlike that due to water pressure, increases towards the springing closely proportionate to the inclination of the arch. Thus in Fig. 31 the increase is $\frac{a\,c}{a\,b}$, and the maximum stress will be $·42 \times 1·7 = ·71$ tons, giving a total of $8·2 + ·7 =$ say 9 tons.

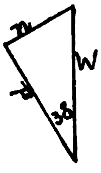


FIG. 30.

**(65)** Considering that in most arched dams the maximum stress is over 20 tons, and in one case is 53 tons, a value for $\lambda$ of 10 tons is by no means excessive. The maximum stress on the base of the buttresses works out to 5¼ tons as follows: $N_1$ in the force polygon Fig. 29 (not shown) measures 7,700 cubic feet $= \frac{7,700}{16} = 481$ tons. Dividing this by the width multiplied by the length of the base or by $3 \times 42$, $\frac{N_1}{b}$ or the mean unit stress $= \frac{481}{126} = 3{\cdot}8$ tons.

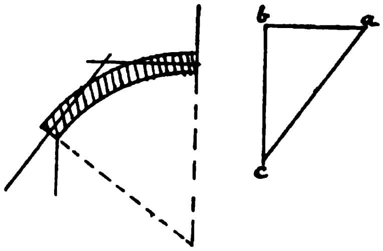


Fig. 31.

The incidence of the centre of pressure being exactly at the ⅔ point of the base the maximum unit pressure will be double the mean or 7·6 tons per square foot. The mean vertical component is 2·6 tons only.

**(66)** The compressive stress in the direction of the axis of the arch, *i.e.*, parallel to the spring line, is exactly normal to the stress in the arch ring induced by external pressure, plus the portion of the weight of the arch transmitted to the skewbacks. This stress has not been considered, being less than that on the radial vertical plane, and acting on a separate plane it cannot be compounded with the other. In the "Principles of Water Works Engineering," this compressive force has likewise been omitted from consideration.

**(67)** The alternative design in Fig. 29 is more efficient and much cheaper than that in Fig. 28, as the following figures will prove:—

Fig. 28: Reinforced concrete in one panel 1,737 feet; this divided by length of panel $= \frac{1737}{8{\cdot}1} = 215$ cubic feet per foot run.

Fig. 29: Plain concrete in one panel 2800 cubic feet divided by length $= \frac{2800}{24} = 178$ cubic feet per foot run.

Thus we see that the alternative design has nearly ⅘ the material of the original work, and there is no necessity for any steel reinforcement. The saving, therefore, would be roughly about 30 per cent.

**(68)** Owing to the curvature of the panels the length of the crest is materially increased, consequently the total length of the weir could be correspondingly reduced, effecting a still further economy. A solid dam would have a sectional area of about 350 square feet, under the proviso of no tension in the concrete. These figures prove the very great economy possible in a weir not submerged from the adoption of the arched panel type of construction.

# CHAPTER III.

## WEIRS (SECTION).

(1) THE hydrostatic conditions affecting overfall weirs differ materially from those that govern the design of the section of dams. In the latter case the water pressure is in one sense only, acting at the back of the wall.

As regards weirs, on the other hand, the water rises at the back till it overtops the crest, and then begins to accumulate in the channel below the weir. This process continues until the maximum flood level of the river or channel is reached.

The velocity of the film of water passing over the crest of the weir is always considerably in excess of that in the river channel itself, consequently the rise of the tail water will always be at a more rapid rate than that of the surface of the film passing over the crest, *i.e.*, of the head water. This variation naturally corresponds with the inverse ratio of the velocities of the film, and of the channel below the obstruction.

(2) The varying pressure to which a weir is subjected may be stated as below.

As the impounded water at the back of the weir fills the reservoir to crest level, the pressure exerted on the weir wall is similar to that in the case of a dam, the opposing forces being (1), the moment of the weight of the water acting normal to the back slope about the base of the wall; (2), the moment of the weight of the wall about its outer toe, or any other point in the base we may think fit to prescribe.

As soon as a film of water passes over the crest new factors come into play. These may be enumerated, in addition, as follows:—(3), the weight of the film supported by the crest of the wall; (4), the reverse pressure normal to the face of the wall exerted by the tail water, which increases from zero at a faster rate than the reciprocal rise of the surface in the upper reach.

In addition to these four forces, when the weir is built on a pervious stratum, as sand, that portion of the weir wall lying below the level of the tail water is subject to loss of weight from flotation, being immersed on three sides in a denser medium than air.

The forces (2), (3) are in favour of the stability of the wall, and so generally, but not invariably, is (4), while force (1), and the reduction in weight due to flotation, militate against it.

(3) When the tail water rises above the crest level of a weir the latter is said to be "drowned" or submerged. Most weirs built at canal heads over

large rivers with sandy beds are submerged, and are further subject to flotation.

Weirs subject to the disability of flotation will first be considered.

The loss of effective weight is equal to that of the water displaced, the material of the wall being surrounded by a denser and heavier medium than that of atmospheric air.

Hence to find the reduced weight of a submerged body we have to deduct from its weight in air the weight of a similar mass of water. As we deal in specific gravities, not in actual weights in tons or pounds, until the conversion into the latter is absolutely required, the reduction in the specific gravity of the material immersed is what is taken into consideration. The specific gravity of water being 1, that of the material minus unity will represent its reduced value. Thus a wall which has a value of $\rho$ of $2\frac{1}{4}$ will, when immersed, have a lowered specific gravity of $2\frac{1}{4} - 1 = 1\frac{1}{4}$, *i.e.*, from $\frac{9}{4}$ to $\frac{5}{4}$, or in the ratio of 5 to 9.

The following table is for the convenience of reference :—

$$\begin{array}{lcl}
\text{When } \rho = 2\frac{1}{2} & \dfrac{\rho - 1}{\rho} & = \frac{3}{5} \\
\rho = 2\frac{1}{4} & ,, & = \frac{5}{9} \\
\rho = 2 & ,, & = \frac{1}{2} \\
\rho = 1{\cdot}8 & ,, & = \frac{4}{9}
\end{array}$$

The reduction of $\rho$ in the effective weight of the weir wall considerably impairs its efficiency as a resisting force, and under certain conditions more than counterbalances the external assistance received by the reverse pressure of the tail water.

(4) By exhaustive graphical tests, which need not all be produced here, the following facts regarding submerged weirs subject to flotation have been elucidated :—

(1) The maximum stress generally occurs when the tail water is at crest level, *i.e.*, just prior to actual submergence.

(2) When the tail water rises to about $\frac{4}{5}$ the height of weir the stress produced by the reciprocal head water is very nearly the same as in the former case.

(3) After the weir wall is once submerged the stress rapidly diminishes. This is due to the fact that no further loss of effective weight due to flotation can take place in the weir wall itself, and the tail water rising at a more rapid rate than the head water, the balance stress becomes less and less, till maximum flood level is reached, when the afflux or difference of level in the water induced by the obstruction is at a minimum.

(5) We have seen in the case of dams that the base width is dependent on the value of $H$, *i.e.*, the depth of water upheld, which is considered as equal to the height of the dam to allow for increase in head due to wave action, and is, with close approximation, $\frac{H}{\sqrt{\rho}}$. *Vide* para. 6, Chap. II.

Very much the same rule is applicable to the required base width for weirs, but in the case of drowned weirs subject to flotation, the height governing the base width is $H + d$, in which $H$ is the height of the weir crest above floor, $d$ the depth of film reciprocal to $D = H$, *i.e.*, the depth of film over crest when the tail water rises to crest level, $D$ being depth of tail water.

With a crest width of $\sqrt{H + d}$, the formula for nearly correct base width is

$$b = \frac{H + d}{\sqrt{\rho}} \text{ or } \frac{H_1}{\sqrt{\rho}} \qquad (1)$$

or with velocity of approach

$$b = \frac{H + d + h}{\sqrt{\rho}} \qquad (1a)$$

This gives a value somewhat in excess of requirements, the resultant falling within the middle third of the base. A closer approximation is obtained by the following :—

$$b = H\frac{(1 + \cdot 9r^{\circ})}{\sqrt{\rho}} \quad (2) \quad \text{or} \quad \frac{h + H\,(1 + \cdot 9r^{\circ})}{\sqrt{\rho}} \qquad (2a)$$

This governing height is a mean between that due to $D = \frac{4}{5}H$ and to $D = H$.

$r^{\circ}$ in this case is the ratio of rise, or $\frac{d}{D}$, in which $D = H$.

(6) In order to obtain the value of $d$, as also of $r^{\circ}$, it will be necessary to make a calculation of the discharge per foot run of the channel below the weir at the level of its crest.

This can be effected by use of the formula $Q = 100\ Ac\ \sqrt{RS}$ (*vide* Chap. V.).

In this expression $A$ is the area of the channel, $c$ a coefficient varying with $R$, with $n$ the rugosity of the channel, and with $S$, and its value is obtained from tables worked out for that purpose. $R$ is the hydraulic mean depth, or the area divided by the wetted perimeter $= \frac{A}{WP}$, and $S$ is the ratio of slope of the bed of the channel.

(7) In the case of submerged weirs over rivers with sandy beds, the base of the weir and its floor often coincide with the average bed level of the river. The river also is of considerable width in proportion to its depth ; consequently the formula $100\ Ac\sqrt{RS}$ can be simplified without sensible error by substituting $D$ for $R$.

As the discharge per unit length, or foot run only, is required, the area $A$ will equal $D \times 1 = D$, and, further, if the friction at the sides of the river be neglected, $WP$, or the wetted perimeter, will equal the base of the water section, *i.e.*, unity, whence $R = \frac{A}{WP} = \frac{D}{1} = D$. The formula then simplified becomes

$$Q = 100\,c\,D^{\frac{3}{2}}\ \sqrt{S}$$

The coefficient $c$ can be obtained from Jackson's "Hydraulic Manual," (Crosby Lockwood). Table X., Part 4, $D$ being taken as $R$. To facilitate calculation the following Table I. gives the formula worked out in terms of $c$ and $D$ for different bed slopes:—

TABLE I.

| $S$ or Slope of Bed. | $S$ per 1,000. | $100\sqrt{S}$. | Discharges per ft. run or $Q$. |
|---|---|---|---|
| 1 in 2,500 | ·4 | 2·00 | $2cD^{\frac{3}{2}}$ |
| 1 in 5,000 | ·2 | 1·414 | $1{\cdot}414cD^{\frac{3}{2}}$ |
| 1 in 6,666 | ·15 | 1·247 | $1{\cdot}247cD^{\frac{3}{2}}$ |
| 1 in 10,000 | ·1 | 1·000 | $cD^{\frac{3}{2}}$ |
| 1 in 20,000 | ·05 | ·707 | $\cdot 707cD^{\frac{3}{2}}$ |

The following Table II. will be useful for purposes of comparison of the effect different slopes and values of $n$ have on the ratio $d : D$:—

TABLE II.

| $S$ = 1 in 2,500 or ·4 per 1,000. | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $n$ = ·0275. | | | | | | | | | $n$ = ·030. | | | | |
| $D$ | $100\sqrt{S}$ | $D^{\frac{3}{2}}$ | $100\sqrt{S}D^{\frac{3}{2}}$ | $c$ | $Q$ | $V$ | $d$ | $\frac{d}{D}$ | $c$ | $Q$ | $v$ | $d$ | $\frac{d}{D}$ |
| 7½ | 2 | 20·54 | 41·08 | ·768 | 31·6 | 4·2 | 4·2 | ·546 | ·711 | 29·2 | 3·9 | 4 | ·533 |
| 12 | 2 | 41·57 | 83·14 | ·826 | 68·7 | 5·7 | 7·1 | ·583 | ·767 | 62·3 | 5·2 | 6·7 | ·56 |
| 15 | 2 | 58·77 | 117·53 | ·850 | 99·9 | 7·0 | 9·0 | ·6 | ·791 | 93·0 | 6·2 | 8·6 | ·573 |
| $S$ = 1 in 5,000 or ·2 per 1,000. | | | | | | | | | | | | | |
| $n$ = ·0275. | | | | | | | | | $n$ = ·0250. | | | | |
| 7½ | 1·414 | 20·54 | 29·04 | ·779 | 22·6 | 3 | 3·5 | ·467 | ·850 | 24·7 | 3·3 | 3·7 | ·500 |
| 12 | 1·414 | 41·57 | 58·78 | ·843 | 49·6 | 4·2 | 5·9 | ·492 | ·914 | 53·7 | 4·5 | 6·1 | 5·1 |
| 15 | 1·414 | 58·77 | 83·10 | ·870 | 72·3 | 4·8 | 7·5 | ·500 | ·942 | 78·3 | 5·2 | 7·7 | ·513 |
| 20 | 1·414 | 89·44 | 126·48 | ·905 | 114·5 | 5·7 | 10·2 | ·510 | ·977 | 123·5 | 6·7 | 10·5 | ·525 |
| $S$ = 1 in 6666 or ·15 per 1,000. | | | | | | | | | | | | | |
| $n$ = ·0275. | | | | | | | | | $n$ = ·0250. | | | | |
| 7½ | 1·247 | 20·54 | 25·61 | ·785 | 20·1 | 2·7 | 3·2 | ·426 | ·856 | 21·9 | 2·9 | 3·4 | ·453 |
| 12 | 1·247 | 41·57 | 51·84 | ·853 | 44·2 | 3·7 | 5·4 | ·450 | ·925 | 48·0 | 4·0 | 5·75 | ·480 |
| 15 | 1·247 | 58·77 | 73·29 | ·883 | 64·7 | 4·3 | 7·1 | ·473 | ·955 | 70·0 | 4·7 | 7·3 | ·490 |
| 20 | 1·247 | 89·44 | 111·54 | ·920 | 102·6 | 5·1 | 9·5 | ·475 | ·993 | 110·8 | 5·5 | 9·8 | ·490 |
| $S$ = 1 in 10,000 or ·10 per 1,000. | | | | | | | | | | | | | |
| $n$ = ·0275. | | | | | | | | | $n$ = ·0250. | | | | |
| 7½ | 1·000 | 20·54 | 20·54 | ·797 | 16·4 | 2·2 | 2·8 | ·373 | ·868 | 17·8 | 2·4 | 3 | ·400 |
| 12 | 1·000 | 41·57 | 41·57 | ·873 | 36·3 | 3·0 | 4·7 | ·392 | ·946 | 39·3 | 3·3 | 5 | ·417 |
| 15 | 1·000 | 58·77 | 58·77 | ·906 | 53·2 | 3·5 | 6·2 | ·413 | ·980 | 57·6 | 3·8 | 6·5 | ·433 |
| 20 | 1·000 | 89·44 | 89·44 | ·949 | 84·9 | 4·2 | 8·4 | ·420 | 1·023 | 91·5 | 4·6 | 8·8 | ·440 |
| $S$ = 1 in 20,000 or ·05 per 1,000. | | | | | | | | | | | | | |
| $n$ = ·0275. | | | | | | | | | $n$ = ·0250. | | | | |
| 7½ | ·707 | 20·54 | 14·52 | ·825 | 12·0 | 1·6 | 2·4 | ·320 | ·899 | 13·1 | 1·7 | 2·4 | ·320 |
| 10 | ·707 | 31·62 | 22·36 | ·819 | 18·3 | 1·8 | 3·2 | ·320 | ·960 | 21·5 | 2·25 | 3·1 | ·310 |
| 12 | ·707 | 41·57 | 29·39 | ·921 | 27·0 | 2·3 | 4·0 | ·333 | ·998 | 29·3 | 2·5 | 4·2 | ·350 |
| 15 | ·707 | 58·77 | 41·55 | ·965 | 40·1 | 2·7 | 5·0 | ·333 | 1·043 | 43·3 | 2·9 | 5·4 | ·360 |
| 20 | ·707 | 89·44 | 63·24 | 1·022 | 64·6 | 3·2 | 7·0 | ·350 | 1·101 | 69·6 | 3·5 | 7·5 | ·375 |

(8) In Table II. the expressions in the final column of Table I. are worked out for different values of $D$ and of $n$, the coefficient of rugosity, by which the discharge $Q$ in column 6 is obtained.

The next column $V$, or the mean velocity of current, is obtained by dividing $Q$ by $D$ as $V = \frac{Q}{A}$ and, as we have seen, $D = A$.

The value of $d$ reciprocal to $D$ is given in the eighth column. This is obtained from the formula for discharge of free overfalls, viz.,

$$= 3{\cdot}333\, d^{\frac{3}{2}} \left\{1 + \left(\frac{h}{d}\right)^{\frac{3}{2}} - \left(\frac{h}{d}\right)^{\frac{3}{2}}\right.$$

which is worked out in Table II., Chap. V. For example, in the third series of Table II. of this chapter, where $S = \frac{1}{10,000}$ and $n = {\cdot}0275$, when $D = 12$, $Q = 36{\cdot}3$, and $V = 3{\cdot}0$. The corresponding value of $d$ is found by examining the discharges per foot run in the seventh column, Table I., Chap V., under $V = 3$. The value of $d$, as will be seen, lies between $4\frac{1}{2}$ and 5 feet, the discharges at which depths are 32·739 and 38·233, the difference being 5·494.

Now the difference between the lower of these two quantities corresponding to $d = 4\frac{1}{2}$ and the given discharge of 36·3 is 3·56, consequently the required value of $d$ will be $4{\cdot}5 + \frac{3{\cdot}56}{5{\cdot}49} \times \frac{1}{2} = 4{\cdot}5 + {\cdot}24$ (nearly) $= 4{\cdot}74$.

(9) The values of $d$ obtained from the above Table II. in this chapter must always be maximum values, and are really only applicable when the widths of the river bed and length of weir are such that the non-inclusion of the sides of the water section in the wetted perimeter has no appreciable effect on the result. The ratio of $d$ to $D$ is dependent on the width of the water section, which latter influences the value of $WP$ and consequently that of $R$. In the following Table III. the discharges are worked out by the ordinary formula of $100\, Ac.\sqrt{RS}$ for two bed widths of 200 feet and 1,000 feet. The length of weir is assumed as equal to the bed width, the side slopes are taken as 1 to 1; consequently the wetted perimeter will be $b + 2\ D\ \sqrt{2} = b + 3D$ nearly.

TABLE III.

| $D$ | $A = (b+D)\,D$ | $WP = b+3D$ | $R = \frac{A}{WP}$ | $c$ | $100\sqrt{RS}$ | $V = 100\sqrt{RS}$ | $Q = AV$ | $\frac{Q}{b}$ | $d$ | ratio $\frac{d}{D}$ |
|---|---|---|---|---|---|---|---|---|---|---|
| $S$ = 1 in 5,000. | | | | | | | | | | |
| $n$ = ·0275 $b$ = 200 | | | | | | | | | | |
| 12 | 2544 | 236 | 10·8 | ·829 | 4·5 | 3·73 | 9489 | 47·44 | 5·7 | ·473 |
| 15 | 3225 | 236 | 12·4 | ·847 | 4·9 | 4·15 | 13383 | 66·90 | 7·1 | ·473 |
| Same with $b$ = 1,000 ft. | | | | | | | | | | |
| 12 | 12144 | 1036 | 11·7 | ·839 | 4·83 | 4·06 | 49256 | 49·25 | 5·8 | ·483 |
| 15 | 15225 | 1045 | 14·5 | ·866 | 5·38 | 4·66 | 70948 | 70·95 | 7·5 | ·500 |

As will be seen, the ratio of $d$ to $D$ falls short of that given in the previous Table, and must vary indirectly with $b$. The former Table, however, is useful as a guide where $b$ exceeds 1,000 feet. When actual velocity observations are taken with a representative water section extending up to highest flood level, the discharge at any level can be obtained with much greater accuracy than by using any formula. Surface velocities should be reduced to mean velocities, and the discharge at any level other than that at which the velocity observations were taken can be obtained, as explained in Chap. V., by using the equation $Q = \frac{ACq\sqrt{R}}{ac\sqrt{r}}$ there given. With the discharge and mean velocity thus obtained, the value of $d$ can be found from Table I., Chap. V., and the ratio $\frac{d}{D}$ or the proportion of the respective rises in the water levels above and below the weir, ascertained. *Vide* also para. 22 (*post*).

(10) Fig. 1 is a diagram of a submerged weir 15 feet high with vertical back, showing three values of $D$, one at $\frac{4}{5}\,H$, one at $H$, and one at $H + 5$, or at 20 feet above base. The corresponding values of $d$ and of $H_1$, *i.e.*, of $(d + H)$, are on the assumption that the slope of the weir bed is ·15 per 1,000, and are approximate to those given in Table II.

Due allowance has been made for the weight of the film of water lying over the crest, the area of which is reduced to a corresponding area of masonry by dividing the depth by $\rho$, *i.e.*, by $2\frac{1}{4}$.

The forces in the polygon 1a are represented by areas, not half-widths, as has hitherto been the practice, the reason being that where the tail water, as when $D = 12$, is below the crest, the weir wall lying below this point has a specific gravity of $(\rho - 1)$, whereas that lying above it has a specific gravity of $\rho$, unimpaired by flotation; also the insertion of the water cap above $H$ is a further reason. According to the procedure adopted in Chap. II., the bases of the triangles of water pressure which are drawn normal to the back and face of the wall, as the case may be, will be $\frac{H_1}{\rho}$ and $\frac{D}{\rho}$ respectively, the areas being thus reduced to equivalent areas of masonry.

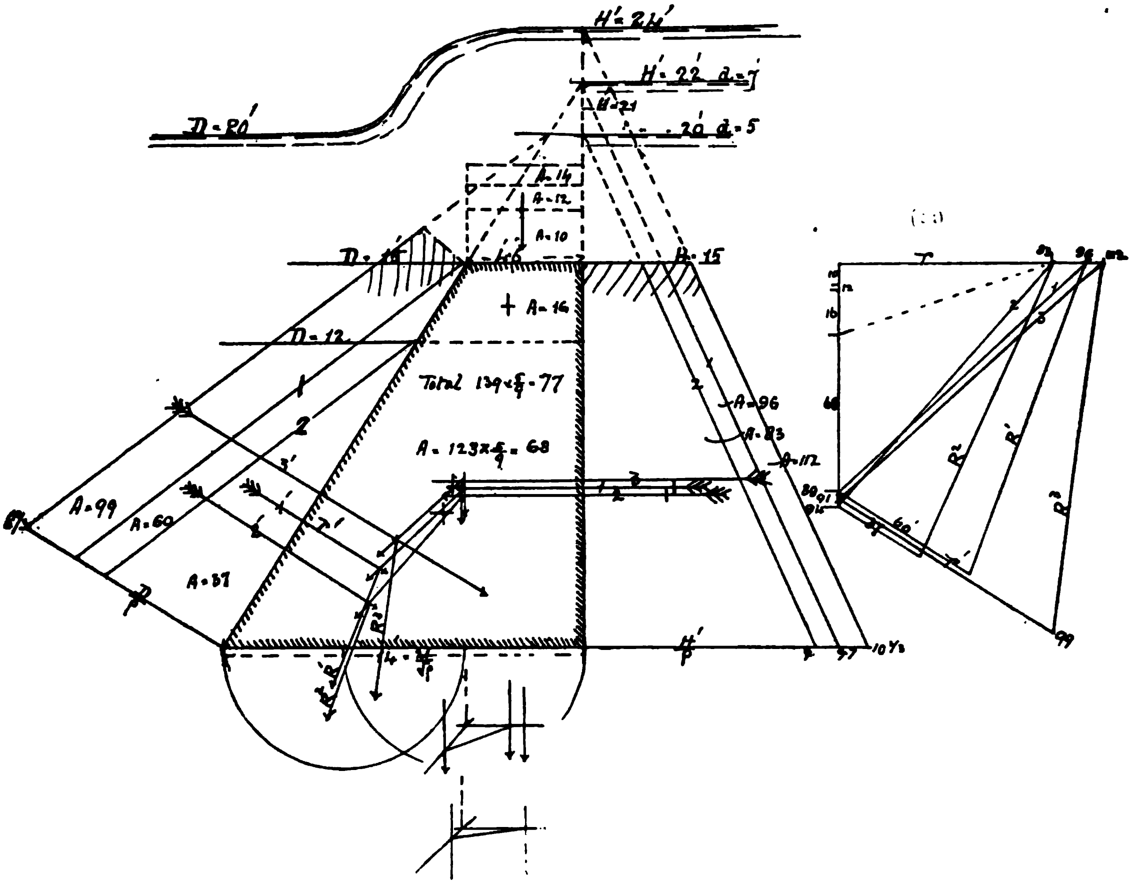
The water pressures are represented by the areas of the trapeziums formed by cutting off the apex of the triangles by lines parallel to their bases at the crest level of the wall, and the weight of the wall is likewise represented by superficial areas, the portion below level of tail water being multiplied by $\frac{\rho - 1}{\rho}$ or $\frac{5}{9}$ to allow for loss of effective weight (para. 3).

The point of application of the resultant pressure of each triangle or trapezium is at the intersection of a line, with the wall back, or face, drawn through its centre of gravity.

In the case of a trapezium being the area of water pressure, this point of application will evidently not be at one-third the height of the wall but at a lower level.

The crest width of the weir wall is made $4\frac{1}{2}$ feet, *i.e.*, nearly $\sqrt{H_1} = \sqrt{22}$. The height governing the base width is taken as a mean between the value of $H+d$ when $d$ is reciprocal to $D = \frac{4H}{5}$, and that of $H+d$ when $D=H$, which is 21 feet. By formula (2) this height will be $\frac{H\,(1 + \cdot 9\, r^{o})}{\cdot \sqrt{p}}$ here $r^{o} = \frac{7}{15}$ ($d$ being 7 feet) and the height $= 15 \times 1\cdot 42 = 21\cdot 3$, which, multiplied by $\frac{1}{\sqrt{p}}$ or $\frac{2}{3}$, gives a base width of 14·2.

Examination of the incidence of the three resultant lines of pressure due



Figs. 1, 1a.

to values of $D = \frac{4}{5}\,H$, $H$ and $H + 5$ shows that $R_1$, due to $D = H$, leads, closely followed by $R_2$ due to $D = \frac{4}{5}\,H$, while $R_3$ due to $D = 20$ and $H_1 = 24$ feet, is far in the rear. Thus, the maximum stress is induced when $D = H$.

The statement commonly made, that a work or weir wall is subject to a head of so many feet of water, can convey no idea of the total horizontal pressure induced, though it states the case with regard to the unbalanced vertical pressure on a horizontal surface, such as the floor of the work. The depth of the water on each side of a work, in addition to the difference in levels, is required to be known before the effect of the unbalanced horizontal pressure on the whole face of the work can be estimated. For instance, the pressure on the wall in Fig. 1 is not represented by the head ($H_1$—$D$), or by the triangle of water pressure having its base at $D$, but by the difference in the areas of the truncated portions of the opposing triangles, which have their bases at the same level as that of the wall, and their apexes at $H_1$ and $D$ respectively. The overturning moments of the opposing forces are represented by the areas of the two triangles or trapeziums of pressure multiplied by the distance of their respective centres of gravity from the base, *i.e.*, the moments would be exactly proportional to the cubes of the depths or as $H^3$ : $D^3$, provided the wall was continuous upward as in the case of a dam, with water on both sides. As it is, however the crest of the wall lies below the apex of some of the triangles of pressure, which latter are truncated forming trapeziums of less superficial area and less moment than the full triangles, the proportion $H^3$ : $D^3$ can be only approximate in a greater or less degree. The case of walls under the disability of flotation still further complicates the problem and renders the adoption of the proportion $H^3 : D^3$ still less accurate, so that graphical trial calculations are required to solve the problem.

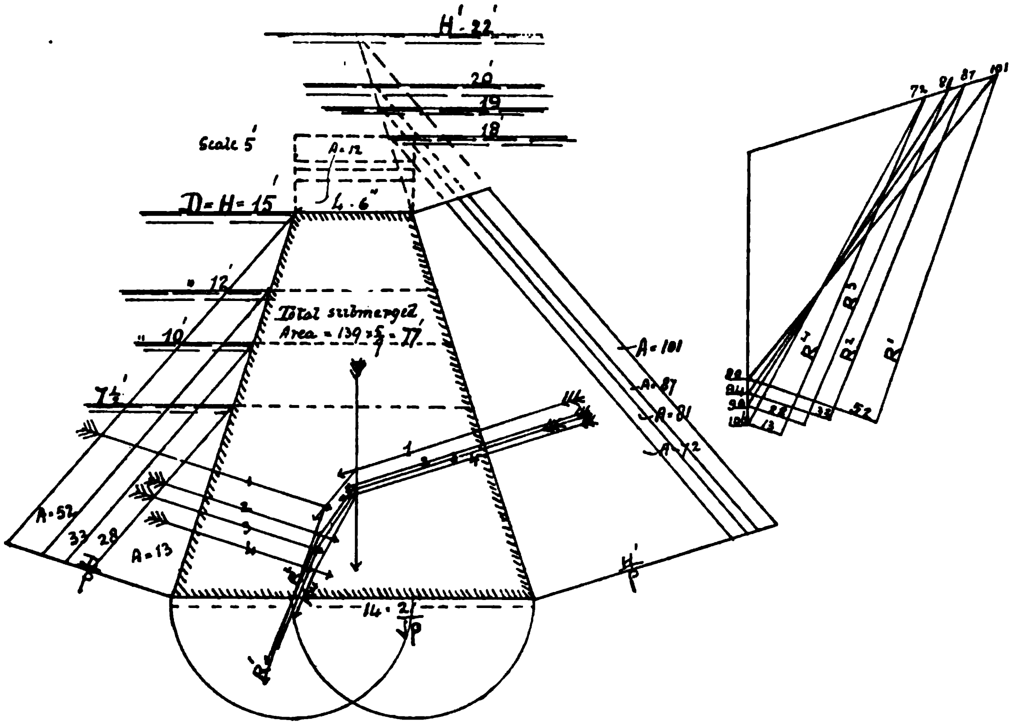


FIG. 2.

(11) In Fig. 2 the section is canted forward to form an equiangular trapezium. This has no apparent detrimental effect on the statical stability of the

wall. Two additional values of $D$ of 10 feet and of $D = \frac{H}{2}$ or $7\frac{1}{2}$ feet are given. All four resultants fall within the middle third.

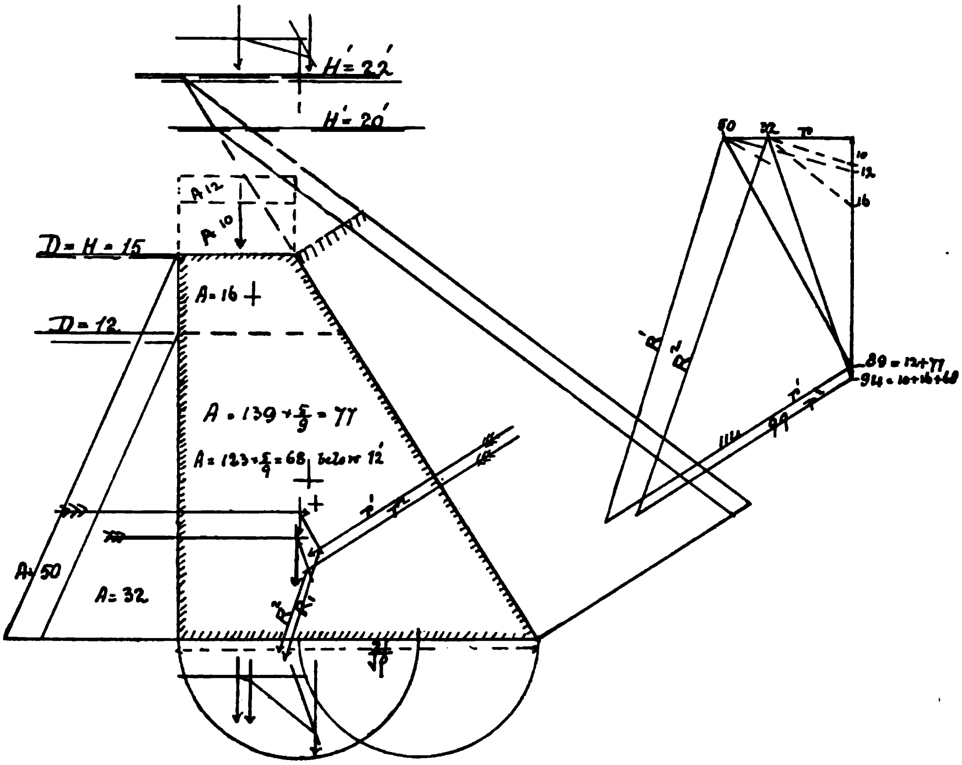


FIG. 3.

(**12**) In Fig. 3 the same wall is shown with section reversed, that is, with a vertical face.

In this case both resultants $R_1$ and $R_2$ fall outside the limiting point, proving that this form is uneconomical, as it will require to be of increased base width and sectional area. In this case the maximum stress is due to $D = \frac{4}{5}H$.

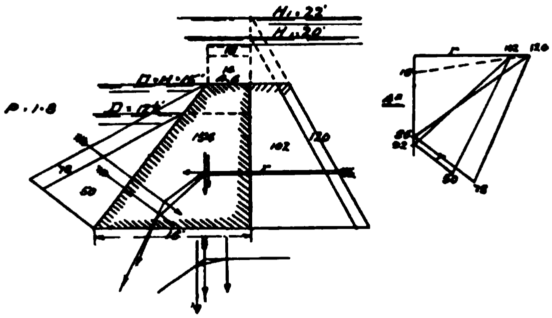


FIG. 4.

(**13**) In Figs. 4, 5, 6, the specific gravity of the wall is assumed at that of brickwork, or $\rho = 1{\cdot}8$. The height governing the base width is taken as 21 feet as before. The value of $\frac{1}{\sqrt{\rho}}$ is $\frac{3}{4}$, instead of $\frac{2}{3}$ as formerly; the base width in all three figures is therefore made 16 feet instead of 14 feet. The increased fore and rear slopes effect some difference in the incidence of the resultants on the base as compared with the last three cases.

In Fig. 4, with vertical back, both resultants fall inside the proper point. In Fig. 5 (the equilateral section) also, the resultant falls within the middle third of the base.

In Fig. 6, with reversed profile, the maximum stress is produced when $D \frac{4}{5} H$; in this case the resultant falls outside the middle third.

The inference to be drawn from these three diagrams is that the vertical or equiangular sections are the best for brick walls of low specific gravity, and that the formula previously given of $\frac{H + d}{\sqrt{\rho}}$ or its modifications will answer equally for brick walls.

With other bed slopes such as 1 in 2,500, 1 in 10,000, or 1 in 20,000, with higher and lower values of $H + d$, the formula $\frac{H + d}{\sqrt{\rho}}$ will still apply as before. Consequently this formula or its modified form (2) is applicable in all cases.

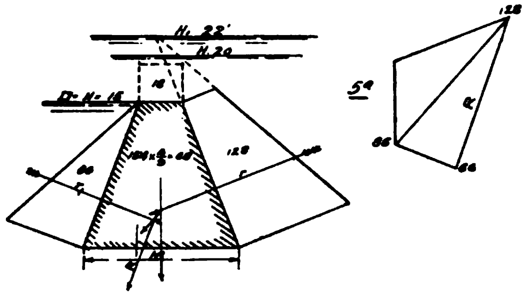


FIG. 5.

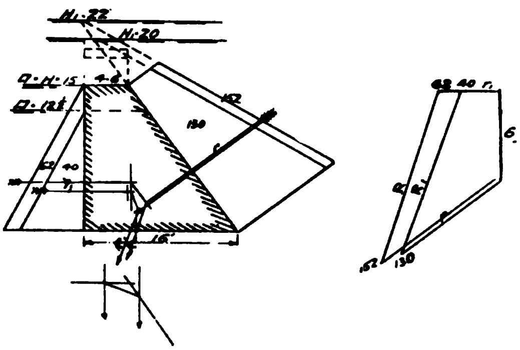


FIG. 6.

(14) Fig. 7 is a model profile for a submerged weir subject to flotation. The base is made $\frac{H + d}{\sqrt{\rho}}$ and the crest $\sqrt{H + d}$. The profile is not equiangular, but the base of the fore slope is to the base of the rear slope as 5 is to 3.

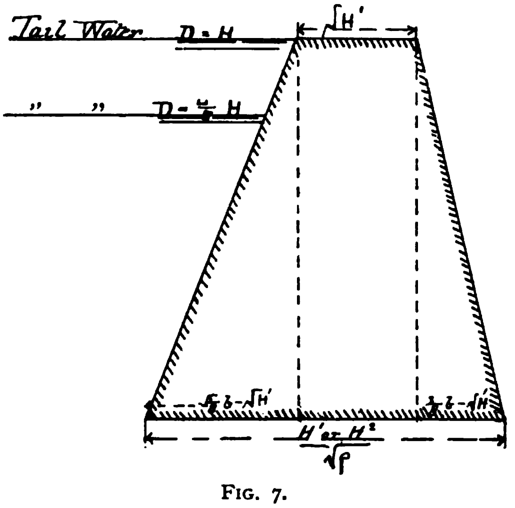


FIG. 7.

(15) In the above calculations for the approximate base width of a weir, the pressure on the weir due to the velocity of approach of the stream has not been taken into

consideration. This velocity is the mean velocity of approach, not that of the actual film passing over the crest.

The depth of the actual film at the weir crest is always less than the depth *d* we have been considering, which latter is the height of the reservoir level some short distance above the weir, measured up from the crest, *i.e.*, before the break in the water level, which always occurs on the current approaching an overfall, causes a diminution in the depth, owing to increased velocity.

The velocity of approach of the current $VA, = Q \div A_1$, in which $Q_1$ is the discharge and $A_1$ the effective area of the water section at the weir. With dwarf weirs on sand $VA$ can be assumed equal to $V$, the mean normal velocity.

Its effect on the weir is that of the corresponding head of water which causes this velocity, and when $V$ is the mean velocity of the stream, this head will be $h = {\cdot}0155 V^2$.

The following table gives the values of *h* due to various mean velocities of approach:—

TABLE IV.

TABLE OF VALUES OF HEAD DUE TO VELOCITY OF APPROACH.

| Mean Vel. $V$. | $V^2$ | Multiplicand. | $h$. | Mean Vel. $V$. | $V^2$. | Multiplicand. | $h$. |
|---|---|---|---|---|---|---|---|
| 2 | 4 | ·0155 | ·06 | 6 | 36 | ·0155 | ·56 |
| 2½ | 6·25 | ” | ·10 | 6½ | 42·25 | ” | ·66 |
| 3 | 9 | ” | ·14 | 7 | 49 | ” | ·76 |
| 3½ | 12·25 | ” | ·19 | 7½ | 56·25 | ” | ·87 |
| 4 | 16 | ” | ·25 | 8 | 64 | ” | ·99 |
| 4½ | 20·25 | ” | ·31 | 8½ | 72·25 | ” | 1·12 |
| 5 | 25 | ” | ·39 | 9 | 81 | ” | 1·40 |
| 5½ | 30·25 | ” |  | 10 | 100 | ” | 1·55 |

These should, strictly speaking, be added to the height $H_1$ or that on which the calculation for the base width is dependent, but for moderate velocities of approach, which mostly prevail when $D = H$, and the bed does not exceed 1 in 10,000, *h* can safely be neglected.

**(16)** The case where a weir is not submerged, but is subject to flotation, is one which hardly ever occurs in practice. If $H_1$ be the actual height of the afflux above the base and $D$ the corresponding maximum depth below, then the formula $\frac{H}{\sqrt{\rho}}$ is slightly insufficient for the base width.

If increased by ·1 $H$ as the former formula $\frac{H + d}{\sqrt{\rho}}$ was decreased, the governing height will be closely approximate to what is required, *b* will then be $= \frac{H + r^{\circ}(D + {\cdot}1H)}{\sqrt{\rho}}$ (3). Where velocity of approach is considered, the numerator will be increased by $+ h$ and if the ratio of rise is not represented truly by $\frac{d}{D}$ the formula will become

$$b = \frac{H + d + \cdot 1 H r^{\circ}}{\sqrt{\rho}} \tag{3a}$$

Where $D$ exceeds $\frac{4}{5} H$, the weir can be considered submerged and formula 1 or 2 then applicable.

(**17**) With regard to the crest width, this is more or less a matter for individual judgment. Most submerged river weirs are now supplied with falling iron shutters which raise the surface of the water during low supply. These are generally about three feet in depth and require a crest width of six feet to manipulate.

Then again weirs subjected to the impact of floating bodies such as rafts, tree trunks, etc., will require a wide crest to withstand such shocks. Usually, however, such articles only come down the river during spates, when the weir, if drowned, is completely protected by the depth of water overflowing the crest. For a general rule that adopted in the model profile Fig. 7, viz., $\sqrt{H_1}$ will be found suitable. It may be increased in high weirs to $\sqrt{H_1} + 1$ or $+ 2$ or be reduced to $\sqrt{H}$.

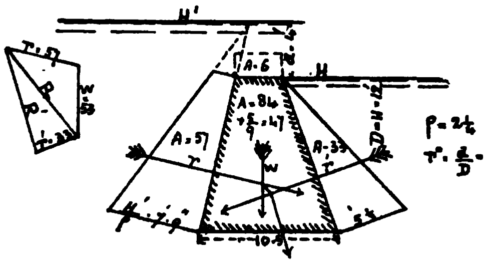


FIG. 8.

(**18**) We will now give two examples of design in accordance with the formulas 2 and 3.

Fig. 8 is the design of the section of a submerged weir under the following probable conditions :—

$H$ or height of weir $= 12$ feet $= D$,
$d$ or depth of film reciprocal to $D = H = 4$ feet,
then $r^{\circ}$, the ratio of $\frac{d}{D}$, will be ⅓, or ·333 feet.

From Table II. we find that with $D = 12$ and $d = 4$ the river bed slope will be about 1 in 20,000 (with $n = \cdot 0275$) and the value of $V$ will be 2·3 feet per second, $h$ therefore will be only ·1 and hence can be neglected.

This being a submerged fall, formula (2) is applicable, viz., $b = H(1 + \cdot 9 r^{\circ})$, $\rho$ being assumed as 2¼, with $\frac{1}{\rho} = \frac{4}{9}$ and $\frac{1}{\sqrt{\rho}} = \frac{2}{3}$, $H$ being 12 feet, we have $b = 12(1 + \cdot 9 \times \cdot 33) \times \frac{2}{3} = 8 \times 1.3 = 10.4$ feet, made 10½ feet.

A top width of $\sqrt{H}$ has been adopted, *i.e.*, $\sqrt{12} = 3{\cdot}5$ and the profile is on the lines of the model Fig. 7.

(19) Another example is given in Fig. 9. In this the weir is not submerged, so that $H_1$ represents the actual afflux flood level, while $D$ is the maximum depth of the river below the weir. The velocity of approach is assumed at 5 feet per second with corresponding value of $h = \cdot 4$ feet. The formula applicable is (3a), or $b = (H + h + d + \cdot 1\, Hr^{\circ}) \frac{1}{\sqrt{\rho}}$. In this case $H = 30, h = \cdot 4, r^{\circ} = \frac{1}{3}$ (assumed), $D = 16$, but does not represent depth of river channel. Then $b = \frac{1}{\sqrt{\rho}} (30\cdot 4 + 4 + 3 \times \frac{1}{3}) = 35\cdot 4 \times \frac{2}{3} = 23\cdot 3$.

In the profile the base is 23·5 feet and the top width 5·5 feet $= \sqrt{H}$; it would have been better if made $\sqrt{H_1}$, *i.e.*, 6 inches thicker.

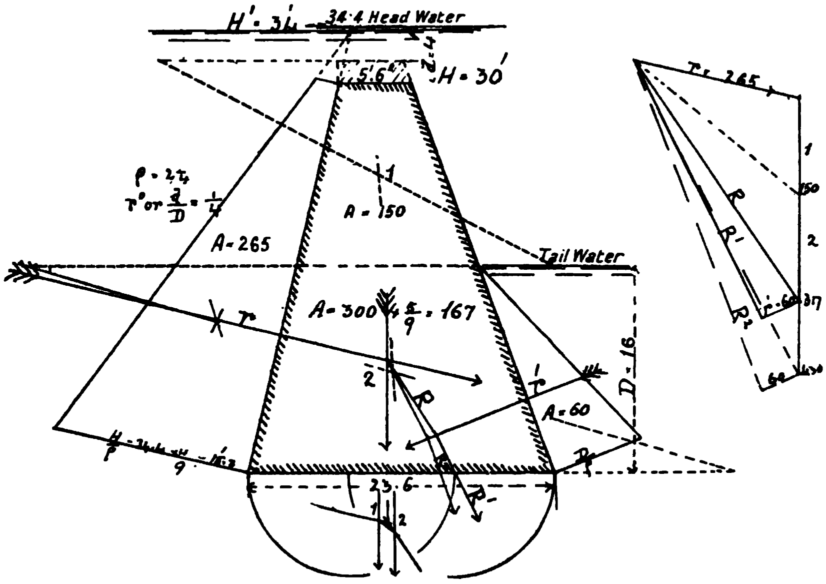


FIG. 9.

The final resultant $R_1$ just passes through the third division of the base. The increase in the crest width to $\sqrt{H + d}$ would bring it within the middle third.

A second resultant $R_2$ shows the altered conditions if the weir body were not subject to loss of weight due to flotation. The base width could then be reduced by one foot.

(20) We will now proceed to consider the case of submerged weir walls founded on rock or clay and consequently not subject to flotation. In this case the equiangular profile will alone be considered, as it is evident that the increase in the weight of the weir will not affect the general conditions except as regards a narrowing of the base.

Fig. 10 is one of many examples that have been worked out. A bed slope of 1 in 5,000 has been assumed and the reciprocal values of $d$ obtained from Table II. for two values of $D$, viz., $D = H$ and $D = \frac{H}{2}$.

The base has been made 13 feet wide or $\frac{19\frac{1}{2}}{\sqrt{p}}$. Both resultants fall within the middle third of the base: $R_2$, due to $D = \frac{H}{2}$, if anything leading. It may be taken as a general fact that the maximum stress on any weir wall not subject to flotation occurs when the tail water level is at, or about, half the height of the weir above its base. The height governing the base width will, however, be greater than that of $H + d$ where $d$ is reciprocal to $D = \frac{H}{2}$. The correct value of $H_2$, the height governing the base above the crest, lies somewhere above half the value of $d$, when the latter is reciprocal to $D = H$, and as a close approximation this value of $d_2$ will be that reciprocal to $D = \frac{3}{5} H$, or $d_2 = \cdot 6d$, and $b = \frac{H + \cdot 6d}{\sqrt{p}}$ (4)

Inspection of the ratios of $d$ to $D$ in Table II. will show that the ratio increases with an increased value of $D$. Thus in the series of $S = 1$ in 10,000, when $D = 7\frac{1}{2}$, $d = 2\cdot 8$. Again when $D = 15$, $d = 6\cdot 7$, if the ratio were even, $d$ would equal $5\cdot 6$. Thus the value of $d_2$ will be higher than half that of $d$ reciprocal to $D = H$. In Fig. 10, $d$ reciprocal to $D = H$, is $7\frac{1}{2}$, and the height governing the base has $d = 4\frac{1}{2}$, which is about reciprocal to $d = 9$ feet.

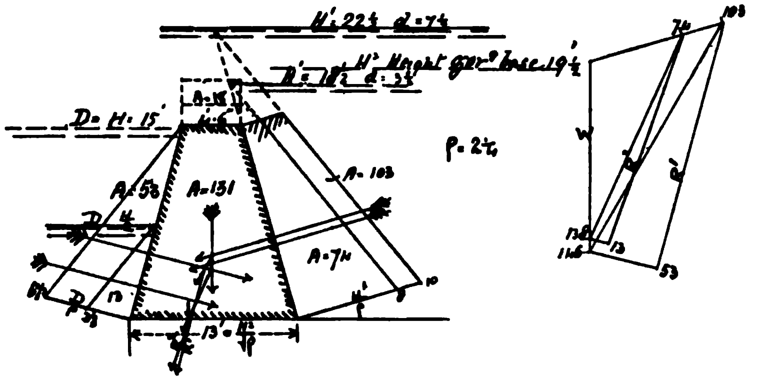


FIG. 10.

The same rule will naturally apply when a weir is not submerged and whatever the incidence of the tail water be on the face.

(21) The slope of bed being known, the discharge of the river per foot run at $\frac{3}{5} H$ has to be calculated or obtained approximately from Table II. Then the corresponding value of $d_2$ obtained from Table II., Chap. V. The base width will then be with close approximation $\frac{H + d_2}{\sqrt{p}}$ or, expressed in terms of $H$ and $r^\circ$ (the latter being the ratio of rise of the head and tail water at $\frac{3}{5} H$),

$$b = \frac{H\,(1 + \frac{3}{5}r^\circ)}{\sqrt{p}} = H\,\frac{(1 + \cdot 6r^\circ)}{\sqrt{p}} \qquad (4a)$$

with velocity of approach $b = \dfrac{H\ (1 + {\cdot}6\ r^{0}) + h}{\sqrt{p}}$ (4b)

When $D$ is less than $\dfrac{H}{2}$ to any considerable extent the height governing base width will be simply $H_1$, and $b = \dfrac{H_1}{\sqrt{p}}$ (5)

(**22**) When deep channels occur in the water section or the bed is very uneven, $D$, the depth at any part will not indicate the mean depth, and the ratio $r^{0}$ will not be the fraction $\dfrac{d}{D}$. In such cases the ratio of rise of head and tail water can only be ascertained by working out the river discharges at two suitable levels of the tail water. The values of $d$ reciprocal to these levels can then be obtained from Table II., Chap. V., and the required ratio will be $\dfrac{d - d}{l - l_1}$, $l$ and $l_1$ in formula being the levels adopted of the tail water.

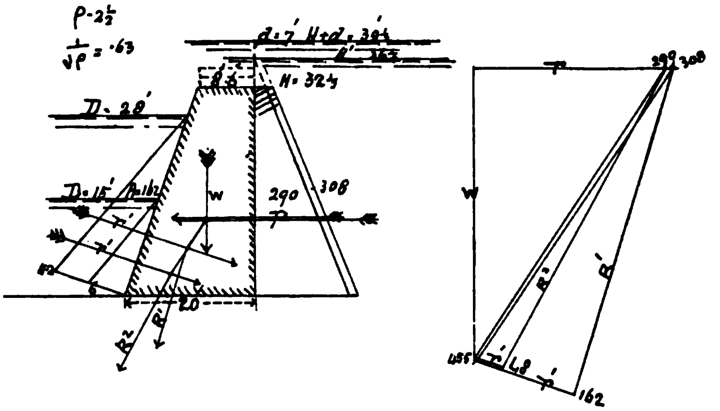


FIG. 11.

(**23**) Fig. 11 is a section of the Nira Weir in India. The crest is 32½ feet high the depth of film $d$ is 7 feet and the depth of tail water, or $D$ is 28 feet. The probability in this case is that the weir base is sunk below the average bed level of the river, or else a subsidiary weir exists below the work which raises the tail water. With a value of $d$ of 7 feet, $D$ cannot well be more than 20 feet. The specific gravity of the material, or $\rho$, has been taken at 2½. The graphical calculations show that under existing flood conditions, with the afflux level at 39½ feet and $D = 28$ feet, the resultant line of pressure passes just within the middle third of the base. When, however, a lower value of $d$ is taken reciprocal to $D = 15$ feet, the line of pressure falls well outside the middle third. This proves conclusively the error of the commonly received doctrine that the maximum stress necessarily occurs at flood levels, on which principle this weir section has evidently been designed.

(24) Fig. 12 is the revised section. The height governing the base width is taken as that corresponding to $D = \frac{5}{8} H$, as far as can be judged, which gives a base width of 24 feet. One resultant is drawn on the profile, of the pressure when $D = \frac{H}{2}$ or 16 feet. This falls just within the middle third.

The top width is reduced to 6½ feet, or $\sqrt{H_1}$.

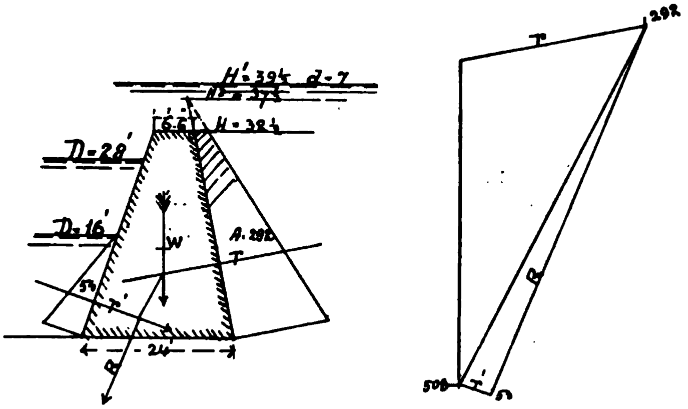
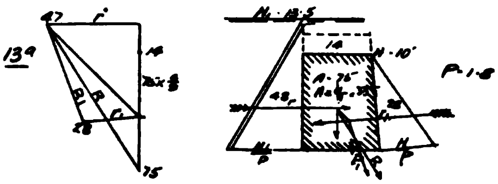


FIG. 12.

(25) If the top width be preserved uniform throughout, the profile designed for the extreme depth will suit for any less depth, but there will be a waste of material. It would be better to keep the top and base width variable, with the face at a uniform batter throughout, and the rear a variable winding batter, in the construction of which, with the help of a few templates, no difficulty whatever will occur.

(26) An example of an existing river weir subject to flotation is given in Fig. 13, which is a section of the Narora Weir at the head of the Lower Ganges Canal. As will be seen from the figure, the weir wall is entirely submerged in flood time. The crest is 10 feet high above floor, which latter corresponds with the normal bed of the river; $H$ therefore will actually represent the depth $D$ of the channel, when the surface has risen to crest level. The value of $d$ reciprocal to $D$ is estimated to be 3½ feet. $H_1$ is therefore 13·5 feet.



FIGS. 13, 13a.

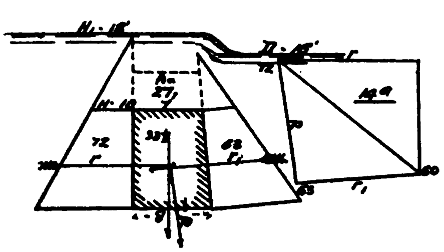
The incidence of $R_1$, the resultant line of pressure due to $H_1 = 13·5$ and $D = 10$, is well without the middle third of the base, proving the insufficiency of the base width. The statical head is 13 feet, not 10 feet, owing to collapsible iron shutters fixed on the crest.

The resultant line $R$ due to this head is also shown on the figure, and this likewise falls outside the middle third.

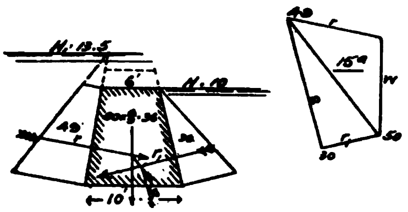
(**27**) In Fig. 14 the resultant line of pressure due to the full flood level of the river is drawn on the section. In this case it cuts the base line at the proper point, giving the impression that the weir section was designed to meet the minimum instead of the maximum water pressure to which it could be subjected.

In Fig. 15 the correct section is given, which has a base width of 10 feet against 8 feet of the actual structure.

(**28**) In dwarf weir walls such as the one just analysed, it would seem that the proviso that there be no tension in the masonry could well be abrogated, as has been done in this case, without any practical statical disadvantage, and in the case of a weir three-quarters of a mile in length considerable economy undoubtedly results from the reduced section thus adopted. On the other hand, as will duly be noticed in Chap. VI. on "River Diversion Weirs," at this particular point at the head of the work, weight in the weir wall is a great desideratum. When the floor is subjected to the maximum statical head, the weir wall itself is free from the disability of loss of weight due to submergence, and this status is of the greatest assistance in neutralising the head acting below the floor. This matter is clearly proved in many examples given in Chap. VI. Thus reduction in the section of a weir wall will not result in any real economy, as this reduction will have to be made good in the foundations, which are more expensive to construct and have only half the effective weight of a similar quantity placed in the wall. This particular weir floor is deficient in frictional stability, for which the light section of the weir wall is partly responsible (*vide* par. 36, Chap. VI.).



FIGS. 14, 14a.



FIGS. 15, 15a.

(**29**) In the case of weirs built in canals and also occasionally as reservoir waste weirs, the weir wall itself is wholly or nearly so in cutting. Under such conditions the excavation for the weir wall is cut back to a slope, and the hollow prism thus formed at the rear of the weir wall is filled up with puddle. This, if properly done, is absolutely watertight, and should safeguard the wall from any hydrostatic pressure due to the head of water. The wall is then practically in the condition of a retaining wall upholding earth to the canal bed, on which is superimposed the weight of the water in the channel. The common practice in canal falls is to adopt a lighter section of wall than would be admissible under the conditions invariably imposed for walls subject to full hydrostatic pressure, viz., that the centre of pressure on the base line of the weir must fall within the middle third of the base, thus ensuring the absence of any tension in the masonry. At the same time it is evident that the section should be such that, in case it is actually subjected to the full hydrostatic pressure by failure of the puddle protection, it should not overturn. To guard against this, the base must be of such a width that the incidence of the resultant line of hydrostatic pressure falls within the outer toe of the base. This is effected by making the base width two-thirds of that obtained by the formula $b = \frac{H_1}{\sqrt{\rho}}$, or $b = \frac{H_1}{\rho}$. Such a section is termed in this work the "Hybrid" section.

**(30)** Some American weirs present features of considerable interest, principally on account of their unusual height, as well as the apparent absence of solid masonry flooring, its place being often taken by heavy timber crib work filled with rubble, planked over and attached to anchor piles. In any other country a concrete or masonry floor would be much cheaper, in addition to its permanency. In the two following examples, however, the weirs are founded on solid rock.

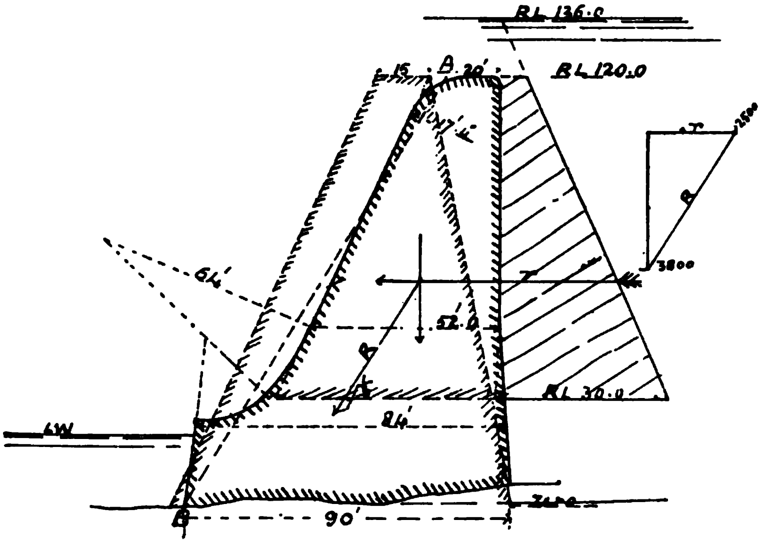


FIG. 16.—La Grange Weir.

Fig. 16 is the section of the La Grange Weir in California, a recently constructed work. Its profile, although good, is not considered absolutely immaculate, and would be improved if the face were altered to the dotted line *AB*. At R.L. 30 the centre of pressure *R* falls outside the middle third of the base. By reducing the excessive top width of 20 feet to 15 feet, and abolishing the excrescence near the base, the section would be statically improved without increase in area. This outline is evidently an improved copy of the obsolete Vrymny weir profile.

The author himself would prefer a plain trapezoidal section as shown in dotted lines, the profile tilted forward to allow the falling water to fall clear of the face of the weir, in a great measure. Some American engineers appear to dread a direct overfall, and the old-fashioned ogee falls, long ago discarded in India, are still clung to in the Western Hemisphere with a pertinacity worthy of a better cause. In a weir of this height the influence of the tail water is practically *nil*, and the approximate formula for base width will be (5), viz., $\frac{H_1}{\sqrt{p}}$, or more correctly $\frac{H+d+h}{\sqrt{p}}$, in this case about 95 feet.

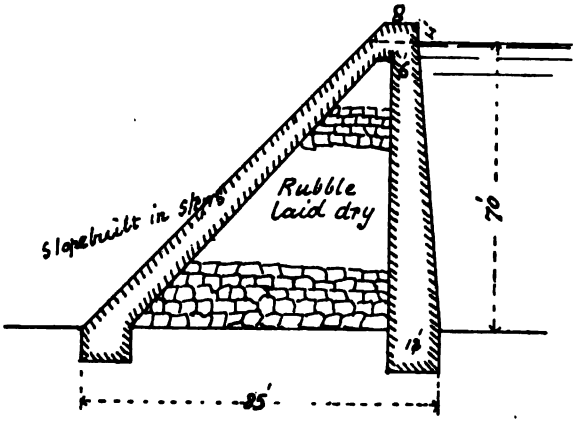


FIG. 17.—Castlewood Dam and Weir.

(31) In Fig. 17, of the Castlewood Weir, we have an excellent and novel style of construction which seems quite irreproachable in point of detail. This weir body is mainly composed of dry rubble masonry, laid without mortar, enclosed by walls of masonry in mortar. This structure has been severely tested. The dam crest is lowered 4 feet for a length of 150 feet and so acts as a waste weir as well as a dam.

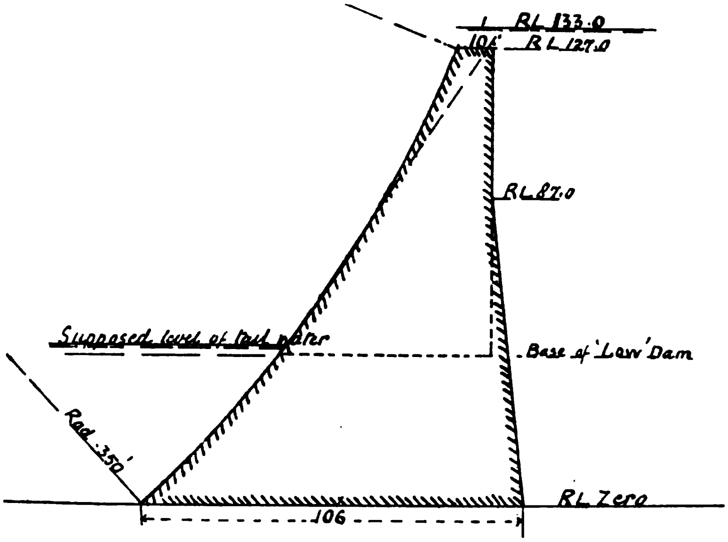


FIG. 18.—Coolgardie Reservoir Weir.

A dam of this type would cost very much less than the usual solid masonry structure, wherever stone is easily available. On the other hand, at sites where stone has to be hauled a considerable distance, the solid dam or weir would probably be more economical. The interstices of the dry masonry should be well filled with spalls and splinters, besides gravel and sand, to raise its specific gravity.

The depth of the tail water is not known, but it cannot be much, as the value of $d$ is only 4 feet.

(**32**) The highest weir in the world is that belonging to the Coolgardie water supply reservoir, near Fremantle, in Western Australia. The profile is given in Fig. 18. On account of its immense height, this structure is to all intents and purposes a dam. The tail water will have very little influence, except detrimentally, as regards load on the foundations.

The limiting stress $\lambda$ is 8 tons. The base of low weir and the elementary triangular profile have been dotted on the section. It is considered that the rear projection of the heel is excessive; a more nearly vertical back would be statically better. If the crest were thrown forward to reduce the fore slope and allow the water to fall clear of the face, the sectional area would have to be somewhat increased.

Several sections of high solid weirs are given in Chap. XIII., of panel weirs in Chap. II., and of dwarf submerged weirs in Chap. VI.

(**33**) A few explanatory remarks concerning the value to be ascribed to the velocity of approach relative to that of the normal velocity of the current, already referred to in para. 15, are deemed desirable before closing this chapter.

The value of the velocity of approach of a river nearing the obstruction caused by a weir is the discharge of the river divided by the effective area of the water section at this spot, or symbolically $\frac{Q}{A_1}$. This area $A_1$ is made up of that of the film passing over the crest of the weir plus that of the depth of water below the crest. This latter depth is liable to be diminished considerably by silt deposit which will impair the value of $A_1$, reducing it, in one case, to that of $A$ the area of the normal water section, or to the extreme limit of that of the area of the film passing over the weir crest only. The value of $d$, *i.e.*, the depth of the film, is also affected by that of $A_1$, of which quantity the former is a factor. So that in any case, the calculation of the area $A_1$ involves a tentative assumption of the value of $d$, and a trial and error process. In deep wide reservoirs the value of $A_1$ is much in excess of $A$, so that the velocity of approach is practically negligible (*vide* also para. 23, Chap. V.).

# CHAPTER IV.

## PIERS, ARCHES, ABUTMENTS AND FLOORS.

(1) INVESTIGATION into the stresses to which piers are subjected is naturally connected with that on arches and abutments, so that these three items will be treated together.

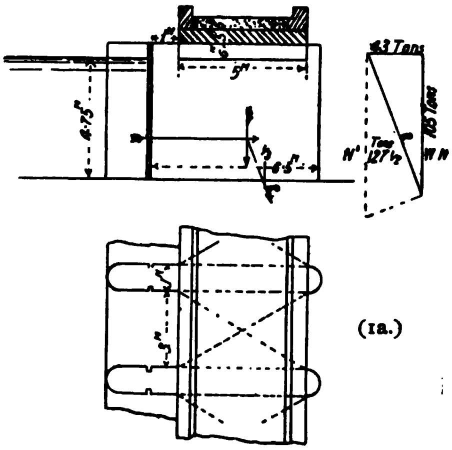
In ordinary bridges of fairly large span the thickness of piers is usually made from one-tenth to one-sixth of the span, and in cases of large spans, say from 40 to 80 feet, the piers should be designed with battering sides so as to produce uniformity of pressure in their masonry. Except in the case of aqueducts, large spans are, however, seldom necessary in irrigation works. Even in road bridges crossing a canal, a series of quite small spans with light arches and piers are far more economical than large spans which, except where navigation is concerned, offer no advantage of any kind. In cross-canal or head regulators, bridged falls, etc., the spans are necessarily small, as, owing to the expense of manipulating heavy regulation gates, these have to be kept within reasonable dimensions. On the other hand, the comparatively recent improvements in non-frictional and counter-weighted roller gates allow of much larger openings being used to what was formerly the practice, but spans of 10 to 25 feet are about the economical limits in such cases.

(2) In irrigation works, piers have not only to safely withstand the weight of the superstructure, and the incidence of stress due to a rolling road, but are often subjected to water pressure, not only in the direction of their length but also cross-wise, from the lateral pressure exerted when one bay of a regulating bridge is closed at the head, being then nearly empty of water, but with a full supply running in the next compartment. This latter cross-stress is a very important item and should never be lost sight of in designing works of this description, as serious failure has occurred from oversight of this point (*vide* par. 44, Chap. II.).

Even when one bay is partially full of water while the next is running full, the pier is often weakened by displacement, for which reason it is very necessary to keep the arches and superstructure well above the highest water level so as to obviate any diminution in their effective weight by flotation.

(3) We will now proceed to investigate the stresses induced in the piers, arches and abutments of a regulating bridge by water, under the most unfavourable circumstances. For this purpose an actual work has been selected.

Fig. 1 is a representation of the transverse section and plan over all of one bay of the Raswaniya Regulator. The dimensions are all in metres. The piers are 1 metre, *i.e.*, $\frac{S}{3}$ in thickness, the spans $S$ being 3 metres wide. Each pier is consequently subjected to the pressure of a prism of water 4 metres in length. The depth being 4·75 metres, the total pressure will be the area of the triangle of water pressure, which is $\frac{h^2}{2}$, multiplied by the length and by $w$ the weight of a cubic metre of water or $\frac{wh^2}{2} \times l$. Taking $w$ as equal to 1 ton, which it is very nearly, we have $r = \frac{(4·75)^2}{2} \times 4 = 43·1$ tons.



FIGS. 1, 1a.

Now the cubical contents of one pier with superstructure, lying to the right of the grooves, is estimated at 60 cubic metres, the weight of which, taking 1·75 for specific gravity of the brickwork, will be 1 × 60 × 1·75 = 105 tons. The portion of the pier upstream of the grooves has been omitted from consideration, as it lies beyond the plane of pressure and is subjected to flotation. Strictly it should have been included in the count.

Fig. 1a is the force polygon. The incidence of the resultant $R$, on the base line, is almost exactly at the outer one-third division of the base. If it fell without to any extent, tension would be set up at the opposite toe, to avoid which the pier would have to be lengthened. The vertical line $N_1$ in Fig. 1a measures the intensity of unit pressure on the masonry at the inter-section of $R$ and equals 127½ tons, which being situated at two-thirds the length of the pier, the maximum at the toe will be $\frac{N_1}{b} \times 2 = \frac{127·5}{6·5} \times 2 = 40$ tons per square metre nearly (*vide* par. 29, Chap. II.), or as a square metre equals 10·75 feet, the pressure per foot square will be $\frac{40}{10·76} = 3·7$ tons. This is by no means excessive. This calculation in the case of large spans with a greater head of water is very necessary, as not only may the pier be in tension at the heel, but the pressure on foundations or floor may be excessive, necessitating a lengthening of the pier down stream to increase its bearing surface.

(4) The thickness of piers is dependent on the weight carried, and consequently is best expressed as some function of the span. The depth of

water regulating the height of the piers is likewise a factor which must not be disregarded, and further the allowable pressure on the foundation, though this latter can be arranged for by widening the pier foundation below the level of the floor base. The thickness may be generally taken as not under $\sqrt{S}$, or to vary from ·36$S$ as a maximum to ·25$S$ as a minimum, for spans from 10 to 25 feet. The greater the span the less fractional proportion required for the pier. Thus, as in Fig. 1, the width of the pier is ·33$S$. For a span of 24 feet under similar conditions the same fraction would give a width to the pier of 8 feet, which is excessive, 6 or 7 feet being a better proportion.

(5) The following table, though naturally empirical, will, it is believed, form a good guide of thickness to adopt in design for partial regulators. The depth of water up-stream of the work forms an additional factor, not the head or horizontal pressure, which latter would be provided for in the length more than in the thickness of the piers.

TABLE I.—TABLE OF PIER THICKNESSES SUITABLE FOR OPEN PARTIAL REGULATORS AND UNDERSLUICES.

| SPAN. | DEPTHS OF WATER. | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 15 ft. | | 20 ft. | | 25 ft. | | 30 ft. | |
| | M | T | M | T | M | T | M | T |
| 10 | ·3 | 3 | ·32 | 3·2 | ·34 | 3·4 | ·36 | 3·6 |
| 12½ | ·295 | 3·7 | ·315 | 3·94 | ·335 | 4·2 | ·355 | 4·4 |
| 15 | ·29 | 4·35 | ·31 | 4·65 | ·33 | 5·0 | ·35 | 5·25 |
| 17½ | ·285 | 5 | ·305 | 5·34 | ·325 | 5·7 | ·345 | 6 |
| 20 | ·28 | 5·6 | ·3 | 6· | ·32 | 6·4 | ·34 | 6·8 |
| 22½ | ·275 | 6·2 | ·295 | 6·64 | ·315 | 7·1 | ·335 | 7·5 |
| 25 | ·27 | 6·75 | ·29 | 7·25 | ·31 | 7·75 | ·33 | 8·25 |

M signifies multiplicand. T thickness.

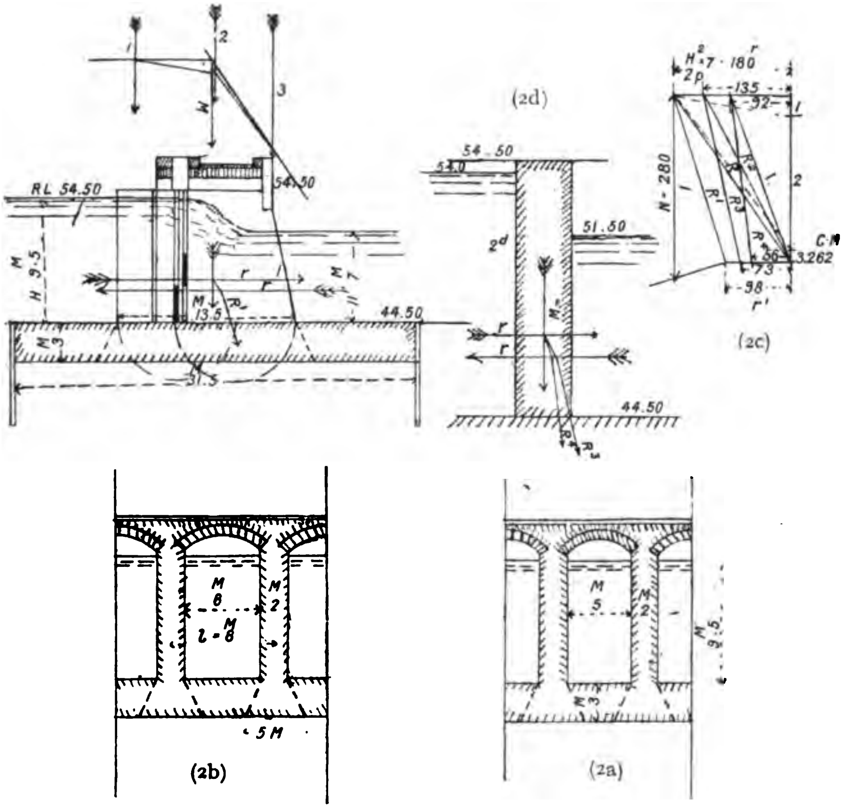
Twenty-five feet is about the outside limit of workable vents for undersluices and similar works whose functions are limited to partial regulation, and in modern practice the least width of vent is hardly under 10 feet. For canal heads subjected to a considerable head of water the thickness of the piers should be ·4$S$.

(6) Inspection of many examples of regulating works will show how far actual practice agrees with the rules for thickness set forth above. Of one quite modern example, viz., the Assiut Regulators, a section is produced in Fig. 2.

The piers in this work are 2 metres in width to spans of 5 metres, giving a ratio of ·4$S$. This proportion appears excessive. There seems to be no reason why the spans should not have been designed of a width of

6 metres or 19¾ feet, retaining the width of the piers at 2 metres; the ratio would then be reduced to ·33*S*, *i.e.*, close upon what is given in the Table for a depth of water of 30 feet, which would be about 6·8 feet to the 6·6 feet of 2 metres. Fig. 2a is a transverse section of the spans as they are, and Fig. 2b as with the suggested enlargement.

The stress diagram is shown in Fig. 2c. The total contents of one span of 6 metres is estimated to be 358 cubic metres. This reduced for flotation will be diminished to 262 cubic metres, $\rho$ being taken as 2. This



FIGS. 2, 2a, 2b, 2c, 2d.—Assiut Regulator.

is the length of the load line *W*. The opposing water pressures are represented by the lines $r$ and $r_1$, each equal to $\frac{H^2}{2\rho} \times l$, viz., 180 and 98 cubic metres ($l$ in this case being 8 metres, the length of one span + the pier). The projection of the resultant $R_1$ on the longitudinal section in Fig. 2 shows the incidence of $R_1$ to be just within the middle third of the base. The question of what water levels up and down stream induce the maximum stress on the piers of a partial regulator is considered in par. 9, Chap. VII.

(7) With regard to lateral pressure, it is deemed that an extreme length of 6 metres is the outside exposed to this force. The values of

$r$ and $r_1$ in this case will therefore be $\frac{6}{8}$ of the former values, $W$ remaining the same. These lengths are marked in the diagram Fig. 2c and projected on to an enlarged section of the pier in Fig. 2d. The final resultant falls just at the pier toe. Thus the pier would not overturn even if not supported at the top by the arching.

Properly speaking, this support should be estimated and the base relieved accordingly. The force $r$ lies $\frac{9·5}{3} = 3·17$, or, say, 3·2 metres above the base, while the distance of $r_1$ is $\frac{7}{3} = 2·33$ metres.

The pier is supported at ten metres height, consequently the proportion of $r$ supported at the top of the pier will be $\frac{3·2}{10} = ·32$, and similarly that of $r_1$ will be $\frac{2·3}{10} = ·23$. Put into figures, the balance pressure in favour of $r$ will be $(·32 \times 135) - (·23 \times 73)$, *i.e.*, $43·2 - 16·8 = 26·4$ cubic metres, equivalent with $\rho = 2$, to $2 \times 26·4 = 52·8$ tons distributed over a length of seven metres of arch, *i.e.*, $\frac{52·8}{7} = 7·54$ tons per metre run, or $\frac{7·54}{10·76} = ·7$ tons per foot run of horizontal superstructure.

To revert to the base, $r$ and $r_1$ will be reduced by $·32r$ and $·23r_1$ respectively, *i.e.*, from 135 and 73 to $135 - 43·2 = 92$ and $73 - 17 = 56$. These reduced values are marked on the force diagram, Fig. 2c, and the resultant line appertaining thereto, viz., $R_4$, projected on to the pier section.

This falls well within the base.

**(8)** Next, with regard to the maximum inclined pressure on the pier section. This is represented in Fig. 2c by the line $N_1$, which measures 280 cubic metres, or 560 tons. Dividing by the length and width of the base, the mean pressure per square metre will be $\frac{560}{13·5 \times 2} = 20·74$ tons; again dividing by 10·76, we obtain mean pressure in tons per square foot, viz., $\frac{20·74}{10·76} = 2$ tons nearly. As the incidence of the resultant is at the boundary of the middle third of the base, the maximum intensity will be double the mean or 4 tons per square foot.

In the original design the piers were carried on circular wells of brickwork sunk in the sand. The width of these was only 2·3 metres, so that, except in length, the bearing surface on the sand was not diminished, but neglecting the weight of the wells themselves and also the skin friction, the load on the actual base would not be less than two tons. The modification of the original design, as was actually executed, is shown in the Figs. 2, mass concrete of one depth throughout of 3 metres, enclosed in iron sheet-piling, having been substituted for a shallower floor and well foundations under the piers.

**(9)** When a pier rests on a mass foundation of this description the load is gradually spread over a larger area, and can be assumed with safety to splay out on each side in correspondence with that usually given to footings at a slope of $\frac{1}{2}$ to 1.

Thus, as shown dotted in Figs. 2, 2a and 2b, the bearing width at the base of the foundation will be the thickness of the pier plus that of the foundation, in both cases $2 + 3 = 5$ metres. The same widening of effective bearing area may be conceived to occur at each end of the piers, increasing the effective length from $13\frac{1}{2}$ to $16\frac{1}{2}$ metres.

The vertical load on the foundation is 358 cubic metres, to which must be added the contents below the floor or a piece $15 \times 5 \times 3 = 225$ cubic metres submerged, *i.e.*, equivalent to 112 cubic metres (with $\rho = 2$). The total will then be $358 + 112 = 470$ cubic metres $= 940$ tons.

This load is distributed over a base $16\frac{1}{2}$ metres long and 5 metres wide, consequently the pressure per square metre will be $\frac{940}{82 \cdot 5} = 11 \cdot 4$ tons, and reduced to per square foot $\frac{11 \cdot 4}{10 \cdot 76} = 1 \cdot 06$ tons. In the case of the actual 5 metre spans, the load will be less by a length of arching of 1 metre, *i.e.*, by some 12 or 15 cubic metres of masonry, which would clearly make no appreciable difference.

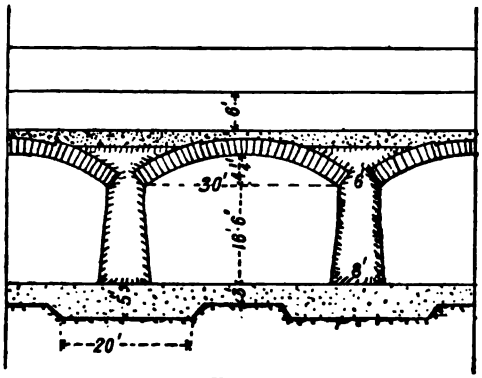


FIG. 3.

**(10)** We gather from this that a load of one ton per square foot is admissible on a foundation of Nile silt, which is of the worst possible kind. On good coarse river sand a unit load of two tons or even over is quite allowable. As regards hard clay, four tons is not exceptional.

In the Nadrai Aqueduct, Fig. 3, Chap. XI., the pressure at the bottom of the well foundations was limited to four tons, and this on pure sand, no allowance, as usual, having been made for skin friction, which latter was ascertained by direct experiment to range from ·6 to $1\frac{3}{4}$ cwt. per square foot of surface. On the Hawkesbury Bridge foundations, also in sand, $4\frac{1}{2}$ tons was the limit pressure. In some works on clay it amounts to over eight tons per square foot. Sand, if protected from erosion, is as good a foundation as any, when of great depth; if, however, the sand is only a thin layer overlying a clay bed, the latter should be reached by protective curtain walls or sheet piling, as otherwise the reduced frictional stability promoted by contact with a smooth hard surface might cause the sand layer to be washed out under hydrostatic pressure (*vide* Fig. 33c, Chap. XIII.).

**(11)** If the sand foundation is efficiently protected by curtain walls or sheet piling, no object is gained by deep foundations, and the same would apply to hard clay in cutting. In the case of large spans where the floor is not sufficiently deep to afford the requisite support to the piers, the foundations of the piers must be carried lower. A good example of how this is effected in mass foundations is given in Fig. 3 of the pier bases of the Budki Superpassage.

The required depth, as we have already seen, should be such that, added to the thickness of the pier, a base width is provided suitable to the limiting pressure on the subsoil.

In regulating works, thick floors to resist the head of water are a necessity which is now thoroughly recognised, and the thickness given is often more than sufficient for pier foundations without anything supplementary in the way of special foundations being supplied underneath the piers.

In the Budki Superpassage the pressure due to the weight of one span, plus that of 6 feet of water carried, is 2¼ tons per square foot at the base of the piers, and 1·45 tons on the ground. The subsoil is believed to be sand.

**(12)** The proportional thickness of piers for aqueducts varies considerably in different examples, as below :—

TABLE II.

| Chap. XI. | Fig. | Span. | Thickness. | Ratio. | $\sqrt{S}$ |
|---|---|---|---|---|---|
| Budki | 8 | 30 | 6 ft. | ·25 | 5·4 |
| Thora Nala | 1 | 30 | 5 ft. | ·166$S$ | 5·4 |
| Kerai | 2 | 20 | 4 ft. | ·2$S$ | 4·4 |
| Gunneram | 4 | 40 | 6 ft. | ·15$S$ | 6·3 |
| Kali Nadi | 3 | 60 | 7 ft. | ·117 | 7·75 |

A good rule for guidance would be to make the top thickness of piers for aqueducts carrying 5 feet or 6 feet of water from 25 feet span upwards $= \sqrt{S}$, below 25 feet $= \cdot 2S$ ($S$ being the span).

Aqueduct piers, like those of large span bridges, do not require to be vertically sided, as is necessarily the case in regulators, where draw-gates in grooves are used, and should always be constructed with either straight or curved battering sides, in order to increase the base width. There is, however, a limit to the batter adopted, as the water-way is thereby diminished. This subject is further treated in Chap. XI.

### *Arches.*

**(13)** The horizontal thrust of an arch is entirely dependent on the weight of that portion of the half arch and its load lying between the crown and the joint of rupture. Whatever weight lies between the joint of rupture and the abutment does not affect the value of the thrust, as it really forms a projecting part of the abutment.

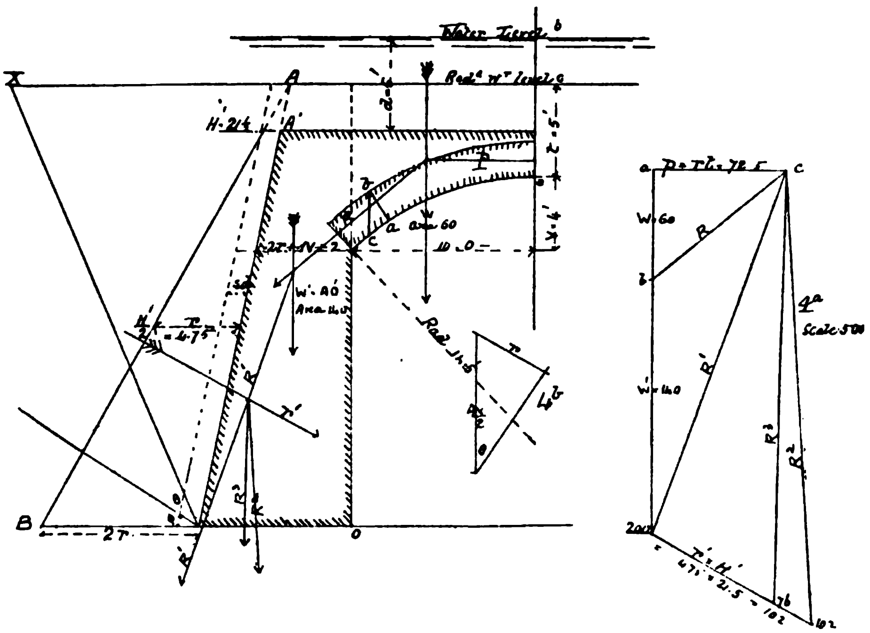
In every arch, of whatever profile, elliptical, segmental or semicircular, the point of rupture is that point where the internal line of pressure tends to go outside the middle third of the arch ring, and so produce tension in the arch. Further, at this point, compared with others, the horizontal stress is at a maximum.

The exact position can easily be found by graphical methods, but as segmental arches are as a rule only employed in modern irrigation works in which the point of rupture is at the springing, it will be superfluous to enter into this question.

The thrust of an arch ($P$) loaded to a horizontal line above the crown is obtained by the following formula with great exactitude:—

$$P = wrt,$$

in which $w$ is the unit weight of the material of the arch ring and filling, the latter reduced to an equivalent area of the material of the arch ring, $r$ the radius of the intrados and $t$ the vertical height of reduced terrain line above the intrados at the crown.



FIGS. 4, 4a, 4b.

(**14**) An example illustrative of the arch thrust on abutments is given in Fig. 4, which represents the segmental half arch of an aqueduct. The spandrel is built up to a horizontal line 6 inches above crown of arch. Above this lies 5 feet of water. This water area is reduced by dividing it by $\rho$, the specific gravity of the material in the arch, and in this case $\rho$ is assumed as 2. The distance $ac$ is thus the $t$ in the formula and equals 5 feet. As the arch and abutment are considered to be of the same specific gravity, $W$, which equals $w\rho$, is a common factor which can be eliminated in the graphical process. The horizontal thrust will then be represented by $rt$, or by $14{\cdot}5 \times 5 = 72{\cdot}5$, and the weights of other parts by their areas. In Fig. 4a $ac = P = 72{\cdot}5$, and the load line $W$, or $ab$, equals the area of the half arch, viz., 60, the resultant being $bc$ or $R$. In the profile Fig. 4, the position of $P$ must be determined with reference not only to its own incidence in the arch ring at the crown, but with reference to that of $R$ in the arch ring in the springing. Both lines must fall within the middle third of the ring, and that position of $P$ which ensures the most axial incidence of both $R$ and $P$ is the correct one.

We now come to the composition of $R$ with the weight of the abutment, which is $W_1$ on the load line, representing the area of the trapezium $AO$. The resultant $R_1$ falls just within the base.

The incidence should be somewhat more recessed within the toe, hence the abutment requires thickening at the base.

### *Thickness of Abutments.*

**(15)** The thickness of an abutment at the spring line of an arch must be such as to ensure against failure by sliding due to the horizontal thrust of the arch. A suitable width is obtained by use of the following empirical formula from "Trantwine"—

$$t = \cdot 2r + \cdot 1V + 2,$$

in which $r$ is the radius of arch and $V$ the versed sine.

In Fig. 4 the thickness is made in accordance with the formula, being $\cdot 2 \times 14\cdot 5 + \cdot 1 \times 4 + 2 = 5\cdot 3$ feet. The rear slope is given by another formula—

Ratio of slope equals $\cdot 02S$ horizontal to $\cdot 5\ V$ vertical

$$\text{or } \frac{\cdot 02S}{\cdot 5V}. \qquad (3)$$

The base width thus obtained will be just sufficient, not counting in the assistance of the earth backing. Under normal conditions of loading these formulas would answer well, but in the case in point the heavy load of water carried would require an increase in the thickness proportional to the depth carried. The increase adopted will be $\cdot 2d$, or one-fifth the depth of water, and the formula would become for aqueducts—

$$t = \cdot 2r + \cdot 1V + 2 + 2d. \qquad (2a)$$

The increased thickness is shown by a dotted line in the profile Fig. 4.

In these calculations the abutment is assumed of the same specific gravity as that of the arch and loading. In stone bridges its specific gravity is greater.

**(16)** The effect of the earth backing is shown in Figs. 4, 4a and 4b. In Fig. 4b, $r$, or the half width of the triangle of earth pressure $AOB$, is obtained by the process frequently employed in Chap. I., the specific gravity of the earth being taken as the same as that of the wall.

The half width $r_1$ of the trapezium of pressure $A_1B$, multiplied by its height, gives its area, and is equal to $4\cdot 75 \times 21\cdot 5 = 102$. By this quantity being set off in Fig. 4a from the extremity of the load line in its proper direction, we obtain $R_2$ on Figs. 4a and 4.

If the earth backing has three-fourths the unit weight of the masonry, the length of $r_1$ in Fig. 4a will be three-fourths less or 76, and the corresponding resultant is $R_3$ in both figures.

### *Thickness of Arches.*

**(17)** There are several empirical formulas giving values of the crown thickness of arches, of which the simplest and best is that in Molesworth's Pocket-book of

$$t = n\sqrt{r} \text{ or } \cdot 4\sqrt{r}. \qquad (4)$$

$n$ is a multiplicand varying from ·4 upwards.

For ordinary bridges a value of ·4 will be ample for the multiplicand, but this should be increased in the case of aqueducts up to ·5 in proportion to depth of water carried (*vide* Chap. XI.), and the formula will then become

$$t = \cdot 5\sqrt{r}. \qquad (4a)$$

This is the thickness adopted in Fig. 4.

In large arches the thickness should increase proportionately with that of the thrust from the crown to the springing. This increase can be obtained graphically as shown in Fig. 4. Here $ab$ is a radial equal to the thickness of the crown, the vertical $bc$ will then be the correct increased thickness at this point. The span in this particular instance being small, no increase is deemed necessary.

### *Buttresses.*

**(18)** Considerable economy can be effected in the section of abutments by the use of buttresses; their effect on the stability of a retaining wall or abutment is obtained as follows:—The wall should be considered as having a base width equal to the normal thickness at base plus the projection of the buttress, but formed of two materials of different specific gravities, the solid portion being of the actual specific gravity of the masonry, and the area behind, which is partly solid buttresses and partly open space, should be considered as of a lighter specific gravity equivalent to that of a material occupying the whole of this rear area, but weighing no more than the buttresses themselves.

The specific gravity of this portion will then be $\rho \times \frac{a}{A}$, in which $a$ is the area of the solid portion, viz., of the projecting buttresses, and $A$ that of the whole space between a line forming the ends of the buttresses and the back of the wall from which they project.

### *Floors, Thickness and Length.*

**(19)** *Firstly: Of Submerged River Weirs on Sand.*

The thickness of the floor should naturally be some function of the maximum statical head.

This, in the case of river weirs provided with falling shutters, will be the height of the weir wall plus that of the shutters. As the latter are usually 3 feet high, the head will be represented by $H + 3$. Even where no shutters are supplied some increase to $H$ (the height of the weir) is required for height governing base to make allowance for sudden freshets and for velocity of approach. The thickness usually adopted in this work is

$t = \sqrt{H + 3} + 1\frac{1}{2}$ for sand foundations. This necessarily empirical rule is founded on actual practice. In case the material of the floor has a specific gravity lower than $2\frac{1}{4}$, as would be the case if brick was the principal material, this value of $t$ should be proportionately increased by the fraction $\frac{9}{4\rho}$, being the average specific gravity of the floor.

For example, let $(H + 3)$ or $H_1 = 16$ and $\rho = 2$. Then $t$ will equal $(\sqrt{16} + 1\frac{1}{2})\,\frac{9}{8} = \frac{99}{16} = 6{\cdot}2$ feet.

The length of submerged river weir aprons on sand foundations is usually made some proportion of $H$, viz., not under either $8\,H$ or $10\,H$, in accordance with the nature and coefficient of frictional stability of the material of the river bed. This subject is fully treated of in Chap. VI. on " River Weirs."

In the above $H$ equals height of crest of weir above base or floor.

(20) *Secondly: Canal Falls.*

The thickness of the floor of canal falls should be $\sqrt{H_1} + 1$, $H_1$ being the height of level of upper reach above the floor, the length being $2H_1$. The same applies to escape falls.

The thickness if on sand to be $\sqrt{H_1} + 1\frac{1}{2}$, and the length is dependent on the coefficient of frictional stability of the sand subsoil and the adoption or otherwise of sheet piling, as will be subsequently explained in Chaps. VII. to IX.

(21) *Thirdly: Undersluices and other partial Regulators on Sand.*

The floors of undersluices are on the same level and subjected to the same hydrostatic pressure as those of river weirs, of which they form an integral part, consequently their thickness will be the same as that given to the weir apron.

If this depth is insufficient for the support of the piers of the superstructure, with a prescribed limiting pressure, it should be increased, either under each pier separately, or else the whole mass can be of increased thickness underneath the sluices, and taper off upwards to the end of the masonry floor, which can be of proportionately less thickness to allow for the excess at the head (*vide* Fig. 10, Chap. VIII.).

The head to which an undersluice is subjected being the same as that on the weir apron, its floor could be made of the same length were it not for dynamical reasons, the rush of water through the sluices, particularly when the river is high, being very great; this length may be put down as $20H_1$.

Of partial regulators, a good example is that of the Assiut Regulators. Although the water is over 30 feet in depth, it is never required to be completely shut off, so that the maximum pressure is due to a head of about 9 feet.

In this case the thickness of the floor is 10 feet, *i.e.*, much in excess of what our formula gives, but this is by reason of the necessity of accommodating the piers. In consequence of this extra thickness at the head, length of floor is saved.

To ensure frictional stability in the sand subsoil, weight is required, and this within reasonable limits can be concentrated at the head of the work provided the total length of creep is also sufficient. This matter is dealt with subsequently in Chaps. VI., VII. and VIII. and Appendix.

The hydro-dynamical force to which these floors (Assiût) are subjected, is nothing like that in Undersluices, so that a very long masonry floor is not a *sine quâ non* as in the latter case.

(**22**) *Fourthly: Head Regulators.*

These works are subjected to greater hydrostatical pressure than any others. Canal Heads, as a rule, have to be entirely closed, even during the highest floods in the river, as often at this very period water is not required in the canal, and a total closure is further necessary for silt clearance and repairs. The Egyptian Canal Heads alone, are partial Head Regulators.

The thickness of the floor on sand should be $\sqrt{H_1} + 1\frac{1}{2}$, subject to increase beneath the superstructure. Thus, with flood level 25 feet above regulator floor, the thickness will be $6\frac{1}{2}$ feet. The length and section of the floor and pitching and the depth of the fore sheet piling are matters entirely of calculation, as will be explained subsequently. Owing to the absence of hydro-dynamic considerations the length of the floor is entirely dependent on frictional stability, induced by its weight superimposed on a porous material.

If a Head Regulator is founded on good clay, the floor thickness need not exceed $\sqrt{H_1}$, and the length be limited to 10 feet to 15 feet beyond the noses of the sluice piers.

For cross-canal Regulators and Branch Heads, the pressure is limited to that due to a very moderate head of water, and a thickness of floor of $\sqrt{D} + 1$ is ample ($D$ being full supply depth), with a minimum of 3 feet. Extension of the masonry floor beyond the down-stream bridge pier noses is hardly required, except when the foundation is porous sand, when the usual calculation for floor area has to be made.

(**23**) The old system of providing water cushions to canal falls by lowering the floor below the bed of the channel down stream involves great and unnecessary expense, in that the height of the weir and retaining walls, in fact all the parts of the structure, have to be increased by the depth of the cushions. In addition to this, the difficulty of getting in the foundations in a water bearing strata is greatly enhanced.

In most cases the water below the channel, which except in notch falls, rises much faster than the level above the fall itself, forms an efficient water cushion; or, if deemed insufficient, a cushion can be provided by building a dwarf subsidiary weir of suitable height across the floor, not at its extreme edge, where the action of the water would tend to damage the pitching on the earthen bed beyond the floor limit, but just sufficiently distant from the crest of the weir to ensure that the falling water drops within it. This distance is $2\sqrt{d \times H}$ where $d =$ depth of film + height due to velocity of approach.

This subsidiary cross wall in canal works should be provided with openings to drain the water out, when the canal is empty. With canal notch falls, the current being split up diminishes materially the velocity of the impact of the falling water.

(**24**) The abolition of water cushions renders it necessary that floors of large falls should be covered by durable stone set in cement mortar. To effect this economically, the surface stones, which should be large and deep, can be laid on ordinary hydraulic mortar and subsequently grouted with cement. The vertical joints will then be all of cement, and with this construction carefully done the impact and swirl of the falling water will have no damaging effect. It is clearly cheaper to do this than to go to the expense of lowering the floor for a water cushion. Ashlar, with fine joints, is the worst possible material for floors. Very exaggerated and erroneous ideas used to prevail regarding the destructive dynamical effect of the impact of falling water, resulting in the design of the Ogee curve profiles in the old Ganges canal works, which eventually had all to be altered to direct overfalls.

# CHAPTER V.

## HYDRAULIC FORMULAS.

(1) THE velocity of a jet issuing from an orifice under the head $h$, is $c\sqrt{2gh}$. (1)

Fig. 1 represents a sluice opening in a dam with a free outlet, *i.e.*, the level of the tail water is below the orifice. In this case $H$ is the mean head, or depth of centre of sluice way below surface; $h_1$ and $h_2$ are the depths to the top and bottom of the orifice, $l$ the width of the orifice whose depth is $h_2 - h_1$, or $d$. Then the discharge will be

$$Q = cl\tfrac{2}{3}\sqrt{2g}\left(h_2^{\frac{3}{2}} - h_1^{\frac{3}{2}}\right) \tag{2}$$

as $\sqrt{2g} = 8{\cdot}025$ and $\frac{2}{3}\sqrt{2g} = 5{\cdot}35$ the formula can be expressed as

$$Q = cl\ 5{\cdot}35\ (h_2\sqrt{h_2} - h_1\sqrt{h_1}) \tag{2a}$$

When $H$ is large compared with $d$

$$Q = \text{(approximately) } cA\sqrt{2gH} \text{ or } cA \times 8{\cdot}025\sqrt{H} \tag{3}$$

A being the area of the orifice or $d \times l$.

(2) In the example given in Fig. 1, $H = 20$, $h_2 = 22$, $h_1 = 18$, $l = 4$ and $c = {\cdot}66$

whence $Q = {\cdot}66 \times 4 \times 5{\cdot}35\ (22 \times 4{\cdot}69 - 18 \times 4{\cdot}242)$ cubic feet.

$= 378{\cdot}8$ cubic feet per second by formula (2a).

The discharge by Formula (3) will be

${\cdot}66 \times 16 \times 8{\cdot}025 \times 4{\cdot}472 = 379$ cubic feet per second.

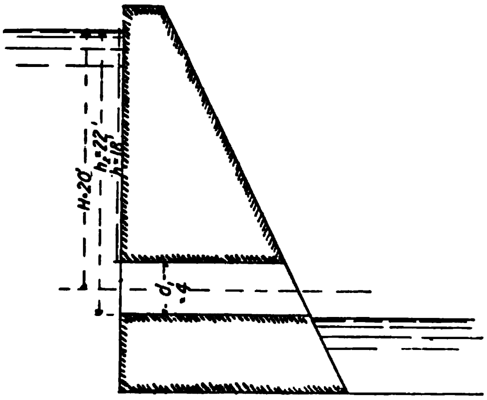


FIG. 1.

Hence in this case the shorter formula, which always gives results somewhat in excess of (2), can well be used.

The velocity of the current through the orifice will be according to (2a)

$$\frac{c \times 5{\cdot}35\ (h_2\sqrt{h_2} - h_1\sqrt{h_1})}{d}$$

or, according to (3), $c\sqrt{2gH}$. In most cases, considering that the suitable value of the coefficient $c$ is by no means known with anything like absolute precision, it would be a useless refinement to adopt the longer and more accurate Formula (2) or (2a).

(3) Fig. 2 represents a similar case in which the sluice or orifice is submerged. Here the head $H$ is the difference in level between the head and tail water.

The exact formula for the discharge through a submerged orifice is

$$Q = cl\,5{\cdot}35\left\{(H\sqrt{H} - h_1\sqrt{h_1}) + \tfrac{3}{2}\ \sqrt{H}\,(h_2 - H)\right\} \qquad (4)$$

In Fig. 2, $H = 16$, $h_1$ and $h_2$ being, as before, 18 and 22 feet,

whence $$Q = {\cdot}66 \times 4 \times 5{\cdot}35\left\{(16 \times 4 - 18 \times 4{\cdot}243) + \tfrac{3}{2}\,(4 \times 6)\right\}$$

$$= 14{\cdot}124\,(-12{\cdot}37 + 36) = 333{\cdot}75 \text{ cubic feet.}$$

The approximate and more commonly used formula for discharge through a submerged orifice is the same as in (3) viz., $Q = cA\sqrt{2gH}$, $H$, however, being the difference between head and tail water, not the depth of the centre of the orifice below the surface. In this instance $Q = {\cdot}66 \times 16 \times 8{\cdot}025 \times 4 = 338{\cdot}98$ cubic feet, the difference being 1·3 per cent. in excess of the last. However, as already stated, in the present unsatisfactory state of hydraulic science the coefficients adopted are only approximate, so that the shorter formula is quite good enough for practical purposes.

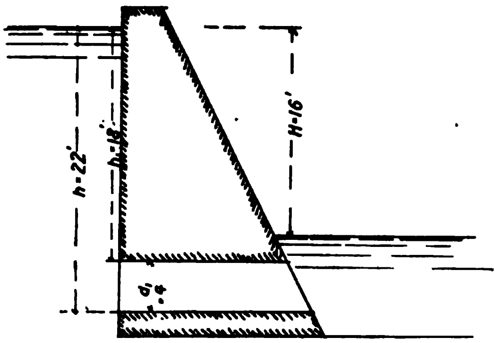


FIG. 2.

In the above cases the same coefficient, ·66, has been used for both free and submerged orifices. As will be seen later, the coefficient is higher when the orifice is submerged.

(4) In the two cases cited the question of the velocity of approach of the current, if any, has not been considered.

Where the section of the orifice is not very small compared with that of the channel of supply, this factor must be taken into account. The velocity of approach can be expressed in terms of the head $h$, expended in producing it, as follows:—

$h = \dfrac{V^2}{2g} = {\cdot}0155\,V^2$; in this $V$ is the mean velocity of the current. If observed surface velocity ($v$) only is known, the mean velocity is commonly assumed to be $V = {\cdot}8v$. A more accurate formula is given in par. 40. The correct theoretical influence of the velocity of approach is best obtained by modifying the coefficient so as to include it, as follows :—

$$c_1 = c\sqrt{1 + \frac{h}{H}} \qquad (5)$$

This obviates two more formulas to replace (2) and (3).

Where the depth of the orifice is small as regards the head of water, the following will give sufficiently accurate results, viz. :—

$$Q = cA\sqrt{2g\,(H + h)} \qquad (6)$$

In this the head due to velocity of approach is simply added to the actual head of water, which is not strictly accurate.

(5) For example, in Fig. 1, supposing the mean velocity of the channel which supplies the sluice to be 3 feet per second, then the modified coefficient will become $c_1 = \cdot 66\sqrt{1 + \frac{h}{20}}$. Now $h$, or the head due to velocity of approach, is, as we have seen, equal to $\cdot 0155 \times 9 = \cdot 1395$, whence $c_1 = \cdot 66\sqrt{1 \cdot 007} = \cdot 662$. The discharge will be increased in the first case of free outfall from 378·8 cubic feet to $378 \cdot 8 \times \frac{662}{660}$ or to 379·95 cubic feet per second.

If Formula (6) be used, the discharge with free outfall will be $\cdot 66 \times 16 \times 8 \cdot 025\sqrt{20 \cdot 1395} = 380 \cdot 31$. With submerged orifice, $H = 16$ and $Q = \cdot 66 \times 16 \times 8 \cdot 025 \times \sqrt{16 \cdot 1395} = 340 \cdot 45$ cubic feet.

These examples are useful as showing the discrepancy between the discharges worked out by the long or by the short formulas, and also showing that when $H$ in either case is below $\frac{d}{5}$ ($d$ being the depth of the orifice or sluice), the longer Formulas (2) and (4) should be used instead of (3) and (6).

(6) In an orifice or sluice way, contraction in the body of water passing through occurs on all four sides. The section of the fluid is thus reduced below that of the opening, and further, the velocity of the water passing is decreased, owing to the friction induced at the sides of the orifice, below what it should be according to theory. A reducing coefficient $c$ has therefore to be used to include both these factors. The exact determination of this coefficient is a matter of great importance. For a circular orifice with thin sides, as a hole cut in a plank, the correct coefficient obtained by experiment is about ·62. As regards rectangular openings, the value of $c$ varies with the shape of the orifice, as it has been found that an oblong hole gives a higher discharge than a square one of the same dimensions under the same head. This proves that the end contractions of the vertical sides have a greater effect than those at the top and bottom of the orifice. Hence a wide shallow orifice gives a higher discharge than a narrow and deep one. The recognition of this fact should influence the design of sluice ways, where it is desirable to reduce the head of water which must form

above the sluice, so as to overcome the frictional resistance of the current in its passage. Cases illustrative of this point are noticed in Chap. XI., which deals with the design of masonry syphons.

(7) The coefficient of discharge through a submerged orifice is always higher than that with a free outfall. The same applies, but in a more marked degree, to cases of overfalls where there is no contraction on the upper surface. Unfortunately, no experiments have been made to ascertain the exact value of the coefficient under various values of $H$, with which it naturally should vary. In "The Principles of Waterworks Engineering," p. 81, the coefficient for a submerged oblong orifice is stated to be experimentally ·67. The ordinary coefficient applicable to small square openings is ·62. In this particular case the opening is 2 feet × 6 inches, *i.e.*, $m$, or the multiplicand $= 1·05$, whence for free outfall $c = ·62 \times 1·05 = ·65$. Deduction from these data would make the coefficient for a submerged orifice under moderate head, $\frac{·62 \times ·67}{·65} = ·639$, or, say, ·64.

In nearly all practical cases the orifice is submerged, which accounts for the high range of many coefficients which have been obtained by testing the discharge through openings on a large scale on actual works, and not on the minute dimensions of orifices and head, from which the original coefficients were deduced.

The rise of the coefficient due to submergence of an orifice shows that up to a certain point the rise in the back water has no influence on the discharge, which is the same as if the orifice were clear. There are no experiments which fix this limit, but it can safely be assumed at $\frac{2}{3}d$, same as in a submerged overfall (*vide*, par. 17).

If $c$ be the proper coefficient for a square orifice, then the multiplicand due to the alteration of the shape to oblong will be as follows :—

when square, $m = 1·00$

when length = twice depth . . . . $m = 1·03$.

„ = 4 times depth . . . $m = 1·05$.

„ = 10 „ . . . $m = 1·07$.

„ = 100 „ . . . $m = 1·08$.

Thus, if the coefficient applicable to any case be assumed as ·8, doubling the width of the opening with same depth as before would increase the coefficient from ·8 to 1·03 × ·8, or ·824.

(8) The following are the coefficients applicable to different classes of orifices or sluices :—

| | $C$ |
|---|---|
| 1. Orifices in a thin plate, such as hole in planks or small iron sluice gates not at base of reservoir, circular pipe outlets, etc. | = ·62 |
| Same submerged . . . . . . . . . . . | = ·64 |
| 2. Sluices without side walls, as tank sluices which open into wide culverts or discharge direct into the lower basin through a dam . . . . . . . . . . . | = ·66 |

3. Sluices in lock gates or moderate sized sluice openings situated at bottom of a reservoir, as many tank sluices are . . = ·84
4. Narrow bridge openings, as canal heads and undersluices of small spans, say, 4 feet to 6 feet, with side walls, bottom sluices in dam . . . . . . . . . . . = ·90
5. Large sluice openings with side walls, as wide undersluice openings of modern type, 15 feet to 20 feet span . . . = ·94
6. Wide openings with bed, level with that of the reservoir, *i.e.*, undersluices or escape heads of exceptional size; also wide bridge openings . . . . . . . . . . . = ·96

**(9)** From the foregoing it will be seen that the value of the coefficient is affected, firstly, by the size of the opening; secondly, by the provision of side walls beyond the sluice gate to guide the issuing current; and, thirdly, by the suppression of the bottom contraction which occurs when the opening is at base of the reservoir, with its floor level with that of the latter.

Thus a sluice built half way up a lock gate will have a coefficient of ·66, while the same if constructed just above floor level will have an increased coefficient of ·84. Where a dam is pierced by sluice openings, as the Bhatgarh and Assuan Dams, the sills of the sluices will not be flush with that of the river, which is uneven; consequently the bottom contraction will exist, and the lower coefficient of ·90 will be applicable. On the other hand, Undersluices in weirs are built on a masonry floor flush or below the normal river bed, and the higher coefficient of ·94 would be applicable. It would be an easy matter to definitely settle the exact suitable coefficients by instituting experiments on a large scale. This could be done with canal Head Regulators, which are of various sizes in many existing works in India. The exact coefficient of discharge could be found by manipulating the gates and noting the resulting level of the canal, from which the discharge could be estimated with great accuracy, and the coefficients obtained under varying conditions of size of aperture and head of water.

The construction of curved piers, noses, or cutwaters at both ends effect a considerable increase in the coefficient in case of a bridge; bluff pier ends, on the other hand, cause a diminution of efficiency of discharge from ·90 to ·86 (*vide* Molesworth's "Pocket Book"). Sharp curved cutwaters are almost as necessary below as above a bridge or sluice way, as they prevent pooling, which is certain to occur where the water section is abruptly enlarged. With regard to the comparative size of orifices, it is clear that the coefficient must vary more or less with the wetted perimeter, *i.e.*, with $\sqrt{R}$, as is recognised in the case of pipes.

**(10)** When water falls over a weir, as in Fig. 3, the conditions are different; the top contraction here is non-existent, and in the equation (2) $h_1 = o$ and $h_2 = d$, the depth of the film, whence the discharge will become

$$Q = \tfrac{2}{3}cl\sqrt{2gd} \times d = 5{\cdot}35\,cld^{\frac{3}{2}} \qquad (7)$$

**(11)** If piers or obstructions are built on the weir, or the weir opening is narrow in proportion to the width of supply channel, the effective length of overfall becomes contracted, and $l_1$ will $= l - {\cdot}1nd$. With pointed cutwaters to piers the deduction should be reduced by $\frac{1}{2}$, and $l_1 = l - \frac{1}{2} {\cdot}1nd$.

In this formula, $n$ equals the number of obstructions, each pier counting as two, and the raised flanks or abutments as one each. This modified value will have to be substituted for $l$ in Formulas (8) and (12).

In a long open weir, where $l$ exceeds $20d$, this end contraction has an inappreciable effect and can be neglected.

**(12)** In free overfalls, the common practice is to adopt Francis Lowell's experimental coefficient of ·623. With this coefficient the discharge of free overfall will be

$$Q = 3{\cdot}333\, ld^{\frac{3}{2}} \qquad (8)$$

This coefficient is generally adopted in Upper India, and is believed to be much nearer the truth than Castel's coefficient of ·666, which has been used by Jackson in the "Hydraulic Manual" and also in "Irrigation Works in India," p. 305, which is practically identical with the former. This latter co-efficient is undoubtedly much too high, except possibly in the case of a very small value of $d$, discharging over a thin plank. A coefficient varying with the depth of film would probably be more strictly in accordance with experiments. These experiments have, however, been undertaken on a very small scale, and a coefficient which may be truly applicable in cases of small depths of 2 feet or 3 feet passing over a narrow plank, the weir opening being likewise a small proportion of the normal width of the supply channel, will not prove so under actual practical conditions.

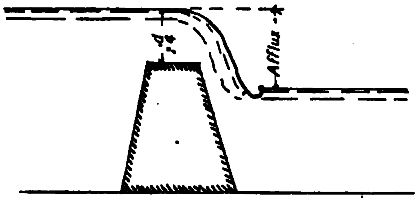


FIG. 3.

The coefficient undoubtedly does decrease to some extent in inverse proportion with the depth of film, but on the other hand with a wide flat-topped masonry weir the increment is in the opposite sense, the fall in one case balancing to a greater or less degree the rise in the other, so that pending further exhaustive experiments on a proper large scale, the adoption of one value of $c$ of ·623 for all depths appears sufficiently accurate for practical purposes.

Besides which, this value is recognised generally as suitable, while that of Castel's, viz., ·666, is not. In the "Madras Manual," the formula having a still lower value of $c$, viz., $Q = 3{\cdot}25d\sqrt{d}$, is adopted.

**(13)** In Fig. 3 the depth of film is shown as 4 feet. The discharge per foot run of weir will then be $Q = 3{\cdot}333 \times 4\sqrt{4} = 26{\cdot}67$ cubic feet.

In this, the velocity of approach, which has always to be considered, except in waste weirs to reservoirs of still water, has been neglected.

The easiest method is to modify the coefficient $c$, using Formula (8). Let

$h$ equal height due to the mean velocity of approach, or $\cdot 0155 V^2$, then this modified coefficient $c_1$ is obtained by use of the following formula:—

$$c_1 = c \times \left\{ \left(1+\frac{h}{d}\right)^{\frac{3}{2}} - \left(\frac{h}{d}\right)^{\frac{3}{2}} \right\} \qquad (9)$$

For example, supposing in the case already cited the mean velocity of the current to be 3½ feet per second, then $h = \cdot 0155 \times (3\cdot 5)^2 = \cdot 19$ feet, and by reference to Table I., in which Formula (9) has been worked out, the multiplicand of the co-efficient with $d = 4$ and $h = \cdot 19$ is 1·062, and the discharge modified to include effect of velocity of approach will be 26·666 × 1·062 = 28·32 cubic feet per foot run.

TABLE I.—MULTIPLICANDS IN FORMULA (9) OF FREE OVERFALLS FOR VELOCITY OF APPROACH, VIZ. $\left(1+\frac{h}{d}\right)^{\frac{3}{2}} - \left(\frac{h}{d}\right)^{\frac{3}{2}}$.

| | MEAN VELOCITIES OF APPROACH. | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 1½ | 2 | 2½ | 3 | 3½ | 4 | 4½ | 5 | 6 | 7 |
| | CORRESPONDING VALUES OF $h$, VIZ.: $\cdot 0155V^2$. | | | | | | | | | | |
| $d$ | ·0155 | ·035 | ·062 | ·097 | ·1395 | ·190 | ·248 | ·314 | ·3875 | ·558 | ·7595 |
| ½ | 1·041 | 1·092 | 1·148 | 1·219 | 1·299 | 1·387 | 1·460 | 1·579 | 1·682 | | |
| 1 | 1·021 | 1·047 | 1·078 | 1·119 | 1·164 | 1·215 | 1·270 | 1·330 | 1·393 | | |
| 1½ | 1·014 | 1·029 | 1·054 | 1·082 | 1·114 | 1·151 | 1·191 | 1·234 | 1·280 | | |
| 2 | 1·011 | 1·024 | 1·040 | 1·061 | 1·089 | 1·117 | 1·158 | 1·182 | 1·224 | | |
| 2¼ | 1·010 | 1·021 | 1·037 | 1·056 | 1·079 | 1·104 | 1·133 | 1·164 | 1·198 | | |
| 2½ | 1·009 | 1·019 | 1 033 | 1·051 | 1·072 | 1·095 | 1·117 | 1·150 | 1·180 | | |
| 3 | 1·008 | 1·017 | 1·028 | 1·043 | 1·061 | 1·080 | 1·102 | 1·127 | 1·153 | | |
| 3½ | 1·107 | 1·014 | 1·024 | 1·037 | 1·052 | 1·070 | 1·089 | 1·111 | 1·134 | | |
| 3¾ | 1·107 | 1·013 | 1·023 | 1·035 | 1·050 | 1·066 | 1·084 | 1·104 | 1·125 | | |
| 4 | 1·006 | 1·012 | 1·021 | 1·033 | 1·047 | 1·062 | 1·079 | 1·097 | 1·118 | | |
| 4½ | 1·006 | 1·011 | 1·019 | 1·029 | 1·041 | 1·055 | 1·071 | 1·088 | 1·107 | | |
| 5 | 1·005 | 1·010 | 1·017 | 1·026 | 1·037 | 1·050 | 1·065 | 1·080 | 1·097 | | |
| 5¼ | 1·005 | 1·010 | 1·016 | 1·026 | 1·036 | 1·048 | 1·062 | 1·077 | 1·093 | | |
| 5½ | 1·005 | 1·010 | 1·015 | 1·025 | 1·034 | 1·046 | 1·059 | 1·073 | 1·089 | | |
| 6 | 1·004 | 1·009 | 1·014 | 1·023 | 1·031 | 1·042 | 1·055 | 1·068 | 1·082 | 1·114 | 1·151 |
| 6½ | 1·004 | 1·008 | 1·014 | 1·021 | 1·029 | 1·039 | 1·050 | 1·063 | 1·076 | | |
| 6¾ | 1·004 | 1·008 | 1·013 | 1·020 | 1·028 | 1·038 | 1·048 | 1·060 | 1·073 | | |
| 7 | 1·003 | 1·007 | 1·013 | 1·019 | 1·027 | 1·037 | 1·046 | 1·059 | 1·071 | 1·099 | 1·134 |
| 7½ | 1·003 | 1·006 | 1·012 | 1·018 | 1·025 | 1·035 | 1·044 | 1·055 | 1·067 | 1·093 | 1·123 |
| 8 | 1·003 | 1·006 | 1·012 | 1·016 | 1·023 | 1·032 | 1·042 | 1·052 | 1·063 | 1·088 | 1·116 |
| 8½ | 1·003 | 1·005 | 1·011 | 1·015 | 1·022 | 1·030 | 1·039 | 1·049 | 1·059 | | |
| 9 | 1·003 | 1·005 | 1·010 | 1·014 | 1·021 | 1·029 | 1·037 | 1·046 | 1·056 | 1·079 | 1·105 |
| 9½ | 1·002 | 1·005 | 1·009 | 1·014 | 1·020 | 1·027 | 1·035 | 1·044 | 1·054 | | |
| 10 | 1·002 | 1·005 | 1·009 | 1·013 | 1·019 | 1·025 | 1·034 | 1·042 | 1·051 | 1·072 | 1·095 |
| 10½ | — | — | 1·008 | 1·012 | 1·018 | 1·023 | 1·033 | 1·041 | 1.049 | 1·068 | 1·091 |
| 12 | — | — | 1·007 | 1·011 | 1·016 | 1·022 | 1·028 | 1·035 | 1·043 | 1·060 | 1·080 |
| 13½ | — | — | — | — | 1·015 | 1·020 | 1·025 | 1·030 | 1·039 | 1·057 | 1·072 |
| 15 | — | — | — | — | — | 1·018 | 1·023 | 1·028 | 1·035 | 1·049 | 1·065 |
| 20 | — | — | — | — | — | — | 1·018 | 1·022 | 1·027 | 1·038 | 1·050 |

**(14)** In case actually observed surface velocities are given in lieu of mean velocities, they can be reduced to mean velocities by use of Formula (17), par. 40, and the results obtained by interpolation from Table I., or else the value of $c$ can be altered by use of Formula (9).

It should be borne in mind that Formula (9) is properly only adaptable for free overfalls where $H$ equals 0, or in special cases where $H$ is ignored.

To use it for submerged falls, substituting $H$ for $d$, as is done in some works, is distinctly wrong.

TABLE II.—DISCHARGES PER FOOT RUN OF FREE OVERFALLS.

$c = {\cdot}623$. (FORMULA) $3{\cdot}333d^{\frac{3}{2}}\left\{\left(1+\frac{h}{d}\right)^{\frac{3}{2}}-\left(\frac{h}{d}\right)^{\frac{3}{2}}\right\}$.

$h = {\cdot}0155V^2$.

| | | MEAN VELOCITIES OF APPROACH IN FEET PER SECOND. | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $d$ | $3{\cdot}333d^{\frac{3}{2}}$ | 1 | 1½ | 2 | 2½ | 3 | 3½ | 4 | 4½ | 5 | 6 | 7 |
| ½ | 1·178 | 1·227 | 1·287 | 1·353 | | | | | | | | |
| 1 | 3·333 | 3·403 | 3·489 | 3·593 | 3·730 | | | | | | | |
| 1½ | 6·123 | 6·209 | 6·301 | 6·454 | 6·625 | 6·821 | | | | | | |
| 2 | 9·427 | 9·531 | 9·653 | 9·804 | 10·002 | 10·266 | | | | | | |
| 2¼ | 11·249 | 11·361 | 11·485 | 11·665 | 11·868 | 12·138 | 12·419 | 12·745 | 13·094 | 13·476 | | |
| 2½ | 13·178 | 13·296 | 13·428 | 13·612 | 13·849 | 14·026 | 14·429 | 14·719 | | | | |
| 3 | 17·319 | 17·457 | 17·613 | 17·804 | 18·064 | 18·375 | 18·722 | 19·085 | 19·518 | 19·969 | | |
| 3½ | 21·824 | 21·977 | 22·129 | 22·348 | 22·631 | 22·959 | 23·352 | 23·766 | | | | |
| 3¾ | 24·204 | 24·373 | 24·529 | 24·760 | 25·051 | 25·414 | 25·801 | 26·237 | 27·721 | 27·229 | | |
| 4 | 26·664 | 26·824 | 26·984 | 27·224 | 27·544 | 27·917 | 28·317 | 28·770 | 29·250 | | | |
| 4½ | 31·817 | 31·976 | 32·166 | 32·421 | 32·739 | 33·121 | 33·566 | 34·076 | 34·617 | 35·221 | | |
| 5 | 37·264 | — | 37·636 | 37·898 | 38·233 | 38·643 | 39·127 | 39·686 | 40·245 | 40·879 | | |
| 5¼ | 40·094 | — | 40·494 | 40·735 | 41·136 | 41·537 | 42·018 | 42·579 | 42·982 | 43·822 | | |
| 5½ | 42·991 | — | 43·422 | 43·636 | 44·066 | 44·452 | 44·969 | 45·528 | 46·129 | 47·075 | | |
| 6 | 48·985 | — | 49·425 | 49·671 | 50·111 | 50·504 | 51·042 | 51·679 | 52·316 | 53·001 | 54·579 | 56·372 |
| 6½ | 55·234 | — | 55·676 | 56·007 | 56·394 | 56·836 | 57·388 | 57·996 | 58·713 | 59·431 | | |
| 6¾ | 58·451 | — | 58·919 | 59·211 | 59·621 | 60·088 | 60·672 | 61·183 | 61·958 | 62·718 | | |
| 7 | 61·728 | — | 62·160 | 62·531 | 62·901 | 63·395 | 64·012 | 64·691 | 65·370 | 66·111 | 67·864 | 70·006 |
| 7½ | 68·458 | — | — | 69·280 | 69·691 | 70·170 | 70·786 | 71·471 | 72·223 | 73·045 | 74·225 | 76·878 |
| 8 | 75·417 | — | — | 76·322 | 76·624 | 77·152 | 77·831 | 78·585 | 79·339 | 80·168 | 82·061 | 84·203 |
| 8½ | 81·597 | — | — | 83·505 | 83·836 | 84·414 | 85·075 | 85·818 | 86·651 | 87·470 | | |
| 9 | 89·991 | — | — | 90·891 | 91·206 | 91·881 | 92·601 | 93·321 | 94·131 | 95·031 | 97·101 | 99·413 |
| 9½ | 97·594 | — | — | 98·472 | 98·960 | 99·545 | 100·23 | 101·01 | 101·88 | 102·86 | | |
| 10 | 105·398 | — | — | 106·24 | 106·77 | 107·40 | 108·03 | 108·98 | 109·82 | 110·67 | 112·96 | 115·44 |
| 10½ | 113·402 | — | — | 114·19 | 114·76 | 115·44 | 116·01 | 117·14 | 118·05 | 118·96 | 121·11 | 123·72 |
| 12 | 138·425 | — | — | 139·44 | 140·00 | 140·68 | 141·44 | 142·33 | 143·27 | 144·38 | 146·80 | 148·02 |
| 13½ | 165·324 | — | — | — | — | 167·80 | 168·63 | 169·46 | 170·28 | 171·77 | 174·75 | 177·23 |
| 15 | 193·638 | — | — | — | — | — | 197·12 | 198·08 | 199·05 | 200·41 | 203·12 | 206·22 |
| 20 | 298·130 | — | — | — | — | — | — | 303·59 | 304·66 | 306·03 | 309·30 | 313·06 |

**(15)** In Table II. Formula (8) is worked out in connection with Formula (9) *i.e.*, Table I. The second column is Formula (8), the items of which are separately multiplied by the proper multiplicands found in Table I. The products fill the remaining columns. For values of $V$ above 7 feet, or $d$ above 15 feet, special calculations will have to be made.

**(16)** Fig. 4 represents a submerged or drowned weir. As in the case of a submerged orifice, the head $H$ is the difference in level of the head and tail water, or the afflux. The depth of the film is termed, as before, $d$, and the submerged portion of the film is $d - H$.

The passing film thus consists of two portions, the upper having a free overfall and the lower being what can possibly be considered as a submerged orifice, but without any top contraction. There can likewise be no bottom contraction or friction in the upper portion. Some authorities, as Jackson, calculate the discharge of each portion with two separate values of $c$, the upper by Formula (8), the lower by Formulas (2) or (3). Jackson uses the coefficient ·66 for the upper and ·62 for the lower. The use of so low a coefficient as ·62 for the lower orifice portion is clearly quite indefensible; it undoubtedly should vary from ·7 to ·9 or more. In the case of an ordinary orifice, the coefficient ·84 given in par. 8 would seem suitable. If, however, a high coefficient be used, the discharge will, in many cases, exceed that of a free overfall, which is clearly impossible — hence the reason of Jackson's adoption of the low coefficient.

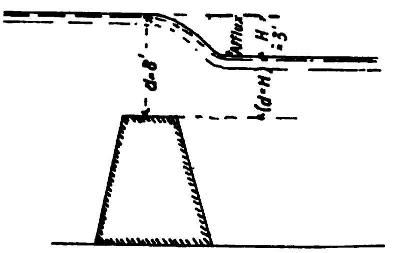


FIG. 4.

But further, cases do occur where, by using this method and these coefficients, the resulting discharge will exceed that of a free overfall. The actual conditions of the two portions of the film are so dissimilar from the hypothetical, that this method of estimating the discharge cannot be satisfactory, and the only alternative is to adopt one coefficient for the whole film, neglecting any imaginary horizontal division in the same.

**(17)** It is clear, however, that the coefficient for the whole film must be a varying one, and the variation must be as $H$ is to $d$. When $H$ is very small in proportion to $d$, the value of the co-efficient will approach unity. This principle has been recognised in the "Madras Manual of Irrigation," where the solution of this difficult question of the single varying coefficient has been attempted. The matter has, however, quite recently been placed on a more satisfactory basis by the results of experiments lately made in Upper India on actual works. These proved the important fact that in a submerged fall when the ratio of $H : d :: 1 : 3$, then the discharge is practically identical with that of a free overfall; and when this proportion is exceeded, then only the discharge begins to be subject to the conditions prevailing in a submerged fall. This apparent paradox is due in part to the depression, or trough caused by the falling water in the tail pond, which neutralises the back pressure of the tail water. On these premises the coefficient can be estimated with some degree of certitude, as it lies between two known values.

**(18)** The theoretical formula for the discharge of a submerged fall, using one coefficient for the whole film above weir crest, is

$$Q = cl \left\{ \sqrt{2gH} \times \tfrac{1}{3}(3d - H) \right\} \qquad (10)$$

taking $c$ and $l$ as unity, and $\sqrt{2g}$ being 8·025, the formula becomes

$$Q = 8{\cdot}025 \sqrt{H} \times \tfrac{1}{3}(3d - H) \qquad (10a)$$

(**19**) When velocity of approach has to be taken into consideration, as is necessary in all cases of running water, as canals and rivers, the formula for discharge is

$$Q = cl\sqrt{2g}\left[(d + h)\sqrt{H + h} - \tfrac{1}{3}\left\{(H + h)^{\frac{3}{2}} + 2h^{\frac{3}{2}}\right\}\right] \qquad (11)$$

This unwieldy formula has been worked out in Table VI., $c$ and $l$ being taken as unity, as it may be found of use for reference, though not employed in this work.

(**20**) We have already noted in par. 17, that experiment has shown that up to a certain depth where $H = \frac{d}{3}$ the discharge is identical with that of a free overfall. The knowledge of this fact enables us to obtain the value of the initial coefficient applicable to Formula (10) when the ratio between the free and drowned portions is that of $H : d :: 1 : 3$. At this ratio the discharge calculated by Formula (8), with $C = {\cdot}623$, must equal that by Formula (10) used with another higher coefficient. By equating these two values, that of the initial coefficient is readily obtained, and is ·8095 nearly.

Thus when $d = 6$ and $V = 0$ the discharge per foot run (*vide* Table II.) is 48·985 feet. In Table III., Series IV., when $H = 2$, $d = 6$ and $c = 1$ the discharge per foot run with $V = 0$, using Formula (10a), is 60·528 cubic feet, whence the initial coefficient required will be $\frac{48{\cdot}985}{60{\cdot}528} = {\cdot}8095$ nearly.

If any other value of $d$ is taken, of the series in which $H = \frac{d}{3}$, the result will be the same. Thus it is clear that taking the coefficient at ·8095 when $d = \frac{H}{3}$, the discharge worked out in the fourth column under $V = 0$ in Table III. will equal that given by Formula (8) for free overfalls for the same value of $d$ in Table II. also under $V = 0$. By this we have now obtained the fixed value of the initial coefficient.

(**21**) With regard to the final coefficient, owing to the want of more definite information it will be necessary to assume some high fixed ratio of $d : H$, which would probably satisfy the conditions of $c =$ unity. Some definite ratio must be assumed, otherwise the problem of the value of the coefficients intermediate between the initial and final values of $c$, or between ·8095 and 1·000 would be indeterminate. The ratio adopted, from various considerations, is $\frac{d}{H} = 100$.

Thus when $H = {\cdot}25$ the initial value of $d$ will be $3 \times {\cdot}25 = {\cdot}75$, and the final value $100 \times {\cdot}25$ feet $= 25$ feet. Again, supposing $H = 2$, then initial $d$ will be 6 feet, and final $d$ 200 feet. In all cases the coefficient will increase with each increase in value of $d$ by a constant, special to each third vertical column in Table III., the unit value of which is $\frac{1 - c_0}{d_1 - d_0}$, equivalent to $\frac{1 - c_0}{(100 - 3)H} =$

$$\frac{1 - c_0}{97H} \qquad (12)$$

In this expression, $c_0 = {\cdot}8095$, the initial coefficient, the final being $1{\cdot}000$; $d_0 = 3H$ is the initial value of $d$, and $d_1 = 100H$ the final value. These constants have been shown on the Table in column for $V = 0$, to enable the values in this column to be extended for any greater value of $d$ than 21 feet, which is the limit in the Table.

For example, in Series III. with $H = 1\frac{1}{2}$

The unit increment ($i$) will be $\frac{1 - {\cdot}8095}{97 \times 1{\cdot}5} = \frac{{\cdot}1906}{145{\cdot}5} = {\cdot}001305$.

Similarly the value of the coefficient applicable to every value of $d$ or of $H = {\cdot}8095 + (\mathrm{d} - d_0)\, i = {\cdot}8095 + i\,(d - d_0)$.

In Table III. the second column contains the discharge worked out according to Formula (10), the third column contains the coefficient and the fourth the product of columns 2 and 3. This last-mentioned column contains the discharges per foot run of weir for each value of $d$, neglecting the velocity of approach.

### TABLE III. (SERIES I.).—TABLE OF DISCHARGES PER FOOT RUN OF SUBMERGED WEIRS.

$H = \frac{1}{2}$.

| | Unit Increment, 5·67451. | Unit Increment, ·00394845 | MEAN VELOCITIES OF APPROACH. | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 0 | 1 | 1½ | 2 | 2½ | 3 | 3½ | 4 | 4½ | 5 |
| | | | MULTIPLICANDS. | | | | | | | | | |
| $d$ | $\frac{1}{2}\sqrt{2gH} \times (3d - H)$ | $c$ | — | 1·014 | 1·029 | 1·054 | 1·082 | 1·114 | 1·151 | 1·191 | 1·234 | 1·280 |
| 1½ | 7·566 | ·8095 | 6·123 | 6·209 | 6·301 | 6·454 | .6·625 | 6·821 | 7·048 | 7·293 | 7·556 | 7·838 |
| 2 | 10·403 | ·811 | 8·437 | 8·55 | 8·68 | 8·89 | 9·13 | 9·40 | | | | |
| 2½ | 13·241 | ·813 | 10·765 | 10·92 | 11·08 | 11·35 | 11·65 | 11·99 | 12·39 | | | |
| 3 | 16·075 | ·815 | 13·101 | 13·28 | 13·48 | 13·81 | 14·18 | 14·60 | 15·08 | 15·60 | | |
| 3½ | 18·912 | ·817 | 15·451 | 15·67 | 15·90 | 16·29 | 16·72 | 17·21 | 17·78 | 18·42 | | |
| 4 | 21·752 | ·819 | 17·815 | 18·11 | 18·33 | 18·78 | 19·28 | 19·85 | 20·51 | 20·97 | 21·98 | 22·80 |
| 4½ | 24·590 | ·821 | 20·188 | — | 20·77 | 21·28 | 21·84 | 22·49 | 23·24 | 24·05 | 24·91 | 25·84 |
| 5 | 27·427 | ·825 | 22·572 | — | — | 23·79 | 24·42 | 25·14 | 25·98 | 26·88 | 27·85 | 28·89 |
| 5½ | 30·264 | ·825 | 24·968 | — | — | 26·32 | 27·02 | 27·75 | 28·74 | 29·54 | 30·81 | 31·96 |
| 6 | 33·101 | ·827 | 27·374 | — | — | 28·85 | 29·62 | 30·50 | 31·51 | 32·60 | 33·78 | 35·04 |
| 6½ | 35·938 | ·829 | 29·792 | — | — | 31·24 | 32·24 | 33·19 | 34·29 | 35·48 | 36·76 | 38·14 |
| 7 | 38·776 | ·831 | 32·223 | — | — | 33·97 | 34·87 | 35·90 | 37·09 | 38·37 | 39·76 | 41·25 |
| 7½ | 41·613 | ·823 | 34·664 | — | — | 36·521 | 37·51 | 38·61 | 39·90 | 41·28 | 42·78 | 44·37 |
| 8 | 44·450 | ·835 | 37·115 | — | — | 39·12 | 40·16 | 41·35 | 42·72 | 44·21 | 45·80 | 47·51 |
| 8½ | 47·288 | ·837 | 39·560 | — | — | 41·72 | 42·83 | 44·09 | 45·56 | 47·14 | 48·84 | 50·66 |
| 9 | 50·124 | ·839 | 42·054 | — | — | 44·32 | 45·50 | 46·85 | 48·40 | 50·09 | 51·90 | 52·83 |
| 9½ | 52·962 | ·841 | 44·544 | — | — | 46·95 | 48·20 | 49·62 | 51·86 | 53·05 | 54 97 | 57·02 |
| 10 | 55·800 | ·843 | 47·040 | — | — | 49·58 | 50·89 | 52·40 | 54·14 | 56·02 | 58·04 | 60·21 |
| 10½ | 57·218 | ·845 | 48·340 | — | — | — | 54·31 | 53·86 | 55·65 | 57·59 | 59·66 | 61·83 |
| 11 | 58·636 | ·847 | 49·665 | — | — | — | 55·74 | 55·33 | 57·16 | 59·15 | 61·29 | 63·57 |
| 12 | 61·474 | ·851 | 52·315 | — | — | — | 56·60 | 58·28 | 60·21 | 62·31 | 64·56 | 66·96 |
| 13 | 64·311 | ·855 | 54·986 | — | — | — | 59·49 | 61·25 | 63·29 | 65·49 | 67·85 | 70·38 |
| 13½ | 65·730 | ·857 | 56·311 | — | — | — | 60·95 | 62·75 | 64·84 | 67·09 | 70·32 | 72·10 |
| 14 | 67·148 | ·859 | 57·680 | — | — | — | 62·41 | 64·40 | 66·39 | 68·70 | 71·18 | 73·83 |
| 15 | 69·986 | ·863 | 60·397 | — | — | — | 65·35 | 67·28 | 69·52 | 71·93 | 74·53 | 77·31 |
| 16 | 72·823 | ·867 | 63·138 | — | — | — | 68·32 | 70·34 | 72·67 | 75·03 | 77·91 | 80·82 |
| 17 | 75·660 | ·871 | 65·900 | — | — | — | 71·30 | 73·38 | 75·85 | 78·48 | 81·32 | 84·35 |
| 18 | 78·498 | ·875 | 68·686 | — | — | — | 74·32 | 76·52 | 79·06 | 81·80 | 84·76 | 97·92 |
| 19 | 81·335 | ·879 | 71·494 | — | — | — | 77·30 | 79·64 | 82·29 | 85·15 | 88·22 | 91·51 |
| 20 | 84·172 | ·882 | 74·240 | — | — | — | 80·33 | 83·70 | 85·45 | 88·43 | 91·61 | 95·03 |
| 21 | 87·009 | ·836 | 77·090 | — | — | — | 83·41 | 85·88 | 88·73 | 91·81 | 95·13 | 98·67 |

## TABLE III. (SERIES II.).—TABLE OF DISCHARGES PER FOOT RUN OF SUBMERGED FALLS.

$H = 1$.

| | Unit Increment, 8·025 | Unit Increment, ·001974 | MEAN VELOCITIES OF APPROACH. | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 0 | 1 | 1½ | 2 | 2½ | 3 | 3½ | 4 | 4½ | 5 |
| | | | MULTIPLICANDS. | | | | | | | | | |
| $d$ | $\sqrt{2gH}\times\frac{1}{3}(3d-H)$ | $c$ | — | 1·008 | 1·017 | 1·028 | 1·043 | 1·061 | 1·080 | 1·102 | 1·127 | 1·153 |
| 3 | 21·400 | ·8095 | 17·319 | 17·457 | 17·613 | 17·804 | 18·064 | 18·375 | 18·722 | 19·085 | 19·518 | 19·969 |
| 3½ | 29·412 | ·810 | 20·584 | 20·75 | 20·93 | 21·16 | 21·47 | 21·84 | 22·23 | 22·68 | 23·20 | 23·73 |
| 4 | 29·425 | ·811 | 23·863 | — | 24·27 | 24·50 | 24·89 | 25·32 | 25·77 | 26·30 | 26·90 | 27·52 |
| 4½ | 33·437 | ·812 | 27·151 | — | 27·61 | 27·92 | 28·32 | 28·81 | 29·32 | 29·92 | 30·60 | 31·31 |
| 5 | 37·450 | ·813 | 30·447 | — | 30·96 | 31·30 | 31·76 | 32·3c | 32·88 | 33·55 | 34·31 | 35·11 |
| 5½ | 41·462 | ·814 | 33·750 | — | 34·32 | 34·68 | 35·20 | 35·81 | 36·45 | 37·19 | 38·04 | 38·92 |
| 6 | 45·475 | ·815 | 37·062 | — | 37·69 | 38·10 | 38·66 | 39·32 | 40·03 | 40·84 | 41·29 | 42·24 |
| 6½ | 42·487 | ·816 | 40·382 | — | 41·07 | 41·52 | 42·12 | 42·84 | 43·61 | 44·50 | 45·51 | 46·56 |
| 7 | 53·500 | ·817 | 43·710 | — | — | 44·93 | 45·59 | 46·44 | 47·21 | 48·17 | 49·26 | 50·40 |
| 7½ | 57·512 | ·818 | 47·045 | — | — | 48·37 | 49·07 | 49·92 | 50·81 | 51·84 | 53·02 | 54·24 |
| 8 | 61·525 | ·819 | 50·389 | — | — | 51·80 | 52·56 | 53·46 | 54·42 | 55·53 | 56·79 | 58·10 |
| 8½ | 65·534 | ·820 | 53·754 | — | — | 55·26 | 56·06 | 57·03 | 58·05 | 59·23 | 60·58 | 61·98 |
| 9 | 69·550 | ·821 | 57·101 | — | — | 58·70 | 59·56 | 60·58 | 61·67 | 62·93 | 64·35 | 65·84 |
| 9½ | 73·562 | ·822 | 60·468 | — | — | 62·16 | 63·07 | 64·16 | 65·31 | 66·64 | 68·15 | 69·72 |
| 10 | 77·575 | ·823 | 63·844 | — | — | 65·63 | 66·59 | 67·74 | 68·95 | 70·36 | 71·95 | 73·06 |
| 11 | 85·600 | ·325 | 70·620 | — | — | 72·60 | 73·66 | 74·93 | 76·27 | 77·81 | 79·41 | 81·42 |
| 12 | 93·625 | ·827 | 77·428 | — | — | 79·60 | 80·76 | 82·15 | 83·62 | 85·32 | 87·26 | 89·27 |
| 13 | 101·650 | ·829 | 84·268 | — | — | 86·63 | 87·89 | 89·41 | 91·01 | 92·86 | 94·97 | 97·16 |
| 14 | 109·675 | ·831 | 91·140 | — | — | 93·69 | 95·06 | 96·70 | 97·80 | 100·44 | 102·72 | 105·08 |
| 15 | 117·700 | ·833 | 98·044 | — | — | 100·78 | 102·26 | 104·03 | 105·70 | 108·05 | 110·50 | 113·45 |
| 16 | 125·725 | ·835 | 104·980 | — | — | 107·71 | 109·49 | 111·38 | 113·38 | 115·69 | 118·31 | 121·04 |
| 17 | 133·750 | ·837 | 111·949 | — | — | — | 116·76 | 118·78 | 120·90 | 123·37 | 126·18 | 129·08 |
| 18 | 141·775 | ·839 | 118·949 | — | — | — | 124·06 | 126·20 | 128·47 | 131·08 | 133·75 | 137·15 |
| 19 | 149·800 | ·841 | 125·982 | — | — | — | 131·40 | 133·67 | 135·75 | 138·83 | 141·98 | 145·26 |
| 20 | 157·845 | ·843 | 133·046 | — | — | — | 138·77 | 141·16 | 143·69 | 146·52 | 149·94 | 153·40 |
| 21 | 165·850 | ·845 | 140·143 | — | — | — | 146·18 | 148·69 | 151·36 | 154·44 | 157·94 | 161·59 |

## TABLE III. (SERIES III.).—TABLE OF DISCHARGES PER FOOT RUN OF SUBMERGED FALLS.

$H = 1\frac{1}{2}$.

| | Unit Increment 9·8258 | Unit Increment ·001305 | MEAN VELOCITIES OF APPROACH. | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 0 | 1 | 1½ | 2 | 2½ | 3 | 3½ | 4 | 4½ | 5 |
| | | | MULTIPLICANDS. | | | | | | | | | |
| $d$ | $\sqrt{2gH}\times\frac{1}{3}(3d-H)$ | $c$ | — | 1·006 | 1·011 | 1·019 | 1·029 | 1·041 | 1·055 | 1·071 | 1·086 | 1·107 |
| 4½ | 39·314 | ·8095 | 31·817 | 31·98 | 32·17 | 32·42 | 32·74 | 33·12 | 33·57 | 34·08 | 34·62 | 35·22 |
| 5 | 44·228 | ·810 | 35·825 | 35·22 | 35·40 | 35·67 | 36·03 | 36·45 | 36·94 | 37·50 | 38·09 | 38·76 |
| 5½ | 49·143 | ·811 | 39·855 | 40·10 | 40·29 | 40·52 | 40·98 | 41·49 | 42·05 | 42·69 | 43·36 | 44·12 |
| 6 | 54·057 | ·811 | 43·840 | 44·10 | 44·32 | 44·68 | 45·11 | 45·64 | 46·25 | 46·95 | 47·70 | 48·53 |
| 6½ | 58·971 | ·812 | 47·884 | — | 48·41 | 48·79 | 49·27 | 49·85 | 50·52 | 51·29 | 52·10 | 53·01 |
| 7 | 63·886 | ·813 | 51·940 | — | 52·51 | 52·93 | 53·45 | 54·07 | 54·80 | 55·63 | 56·51 | 57·01 |
| 7½ | 68·800 | ·813 | 55·934 | — | — | 56·96 | 57·56 | 58·23 | 59·01 | 59·91 | 60·86 | 61·92 |
| 8 | 73·714 | ·814 | 60·004 | — | — | 61·14 | 61·74 | 62·46 | 63·30 | 64·26 | 65·28 | 66·43 |
| 8½ | 78·629 | ·815 | 64·083 | — | — | 65·30 | 65·94 | 66·71 | 67·61 | 68·63 | 69·72 | 70·93 |
| 9 | 83·543 | ·815 | 68·087 | — | — | 69·38 | 70·06 | 7c·88 | 71·83 | 72·92 | 74·08 | ·75·37 |
| 9½ | 88·457 | ·816 | 72·181 | — | — | — | 74·27 | 75·14 | 75·89 | 77·31 | 78·53 | 79·91 |
| 10 | 93·371 | ·817 | 76·284 | — | — | — | 78·50 | 79·41 | 80·48 | 81·70 | 83·00 | 84·45 |
| 11 | 103·200 | ·818 | 84·418 | — | — | — | 86·87 | 87·18 | 89·06 | 90·41 | 91.85 | 93·45 |
| 12 | 113·029 | ·820 | 92·676 | — | — | — | 95·36 | 96·48 | 97·77 | 99·26 | 100·83 | 102·59 |
| 13 | 122·859 | ·821 | 100·866 | — | — | — | 103·81 | 105·01 | 106·19 | 108·03 | 109·74 | 111·92 |
| 14 | 132·686 | ·822 | 109·068 | — | — | — | 117·52 | 118·90 | 120·49 | 122·32 | 124·26 | 126·43 |
| 15 | 142·514 | ·824 | 117·432 | — | — | — | 120·84 | 122·25 | 123·89 | 125·77 | 127·76 | 130·00 |
| 16 | 152·343 | ·825 | 125·683 | — | — | — | — | 130·84 | 132·60 | 134·61 | 136·74 | 139·13 |
| 17 | 162·172 | ·826 | 133·954 | — | — | — | — | 139·36 | 141·32 | 143·47 | 145·74 | 148·29 |
| 18 | 172·000 | ·827 | 142·244 | — | — | — | — | 148·08 | 150·07 | 152·34 | 154·76 | 157·47 |
| 19 | 181·828 | ·829 | 150·735 | — | — | — | — | 156·92 | 159·03 | 161·44 | 164·00 | 166·86 |
| 20 | 191·657 | ·830 | 159·075 | — | — | — | — | 165·60 | 167·82 | 170 37 | 173·07 | 176·10 |
| 21 | 201·486 | ·831 | 167·435 | — | — | — | — | 174·30 | 176·65 | 179·32 | 182·07 | 185·35 |

## Table III. (Series IV.).—Table of Discharges per Foot Run of Submerged Falls.

$H = 2.$

|  | Unit Increment, 11·349 | Unit Increment, ·00098196 | Mean Velocities of Approach. |  |  |  |  |  |  |  |  |  |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
|  |  |  | 0 | 1 | 1½ | 2 | 2½ | 3 | 3½ | 4 | 4½ | 5 |
|  |  |  | Multiplicands. |  |  |  |  |  |  |  |  |  |
| $d$ | $\sqrt{2gH}\times\frac{1}{3}(3d-H)$ | $c$ | — | 1·004 | 1·009 | 1·014 | 1·023 | 1·031 | 1·042 | 1·055 | 1·068 | 1·082 |
| 6 | 60·5283 | ·8095 | 48·985 | — | 49·43 | 49·67 | 50·11 | 50·50 | 51·04 | 51·68 | 53·32 | 53·00 |
| 6½ | 66·203 | ·810 | 53·625 | — | 54·11 | 54·38 | 54·86 | 55·29 | 55·88 | 56·57 | 57·26 | 58·02 |
| 7 | 71·877 | ·811 | 58·292 | — | 59·82 | 59·11 | 59·63 | 60·10 | 60·74 | 61·50 | 62·26 | 63·07 |
| 7½ | 77·552 | ·811 | 62·895 | — | 63·46 | 63·78 | 64·34 | 64·84 | 65·54 | 66·35 | 67·17 | 68·05 |
| 8 | 83·226 | ·812 | 67·580 | — | 69·19 | 68·53 | 69·13 | 69·67 | 70·42 | 71·30 | 72·17 | 73·12 |
| 8½ | 88·901 | ·812 | 72·188 | — | — | 73·20 | 73·85 | 74·43 | 75·22 | 76·16 | 77·10 | 78·10 |
| 9 | 94·575 | ·813 | 76·795 | — | — | 77·87 | 78·56 | 79·18 | 80·02 | 81·02 | 82·02 | 83·09 |
| 9½ | 100·249 | ·813 | 81·503 | — | — | 82·64 | 83·38 | 84·03 | 84·92 | 85·99 | 87·05 | 88·18 |
| 10 | 105·924 | ·814 | 86·222 | — | — | 87·43 | 88·20 | 88·59 | 89·84 | 90·97 | 92·09 | 93·29 |
| 11 | 117·273 | ·815 | 95·577 | — | — | — | 97·78 | 98·54 | 99·59 | 100·71 | 102·08 | 103·42 |
| 12 | 128·622 | ·816 | 104·979 | — | — | — | 107·37 | 108·21 | 109·37 | 110·30 | 112·09 | 113·55 |
| 13 | 139·971 | ·817 | 114·356 | — | — | — | 116·99 | 117·90 | 119·16 | 120·65 | 122·14 | 123·73 |
| 14 | 151·320 | ·818 | 123·780 | — | — | — | 126·63 | 127·62 | 128·68 | 130·59 | 132·20 | 133·93 |
| 15 | 162·669 | ·819 | 133·226 | — | — | — | 136·29 | 137·36 | 138·82 | 140·55 | 142·29 | 144·15 |
| 16 | 174·018 | ·820 | 142·662 | — | — | — | — | 147·42 | 148·65 | 150·51 | 152·36 | 154·36 |
| 17 | 185·367 | ·821 | 152·186 | — | — | — | — | 156·90 | 158·58 | 160·56 | 162·53 | 164·66 |
| 18 | 196·716 | ·822 | 161·700 | — | — | — | — | 166·04 | 168·49 | 170·59 | 172·70 | 174·95 |
| 19 | 208·065 | ·823 | 171·238 | — | — | — | — | — | 178·43 | 180·66 | 182·88 | 185·28 |
| 20 | 208·065 | ·823 | 171·238 | — | — | — | — | — | 188·39 | 190·74 | 193·09 | 195·62 |
| 21 | 230·764 | ·824 | 190·150 | — | — | — | — | — | 198·14 | 200·61 | 203·98 | 205·74 |

## Table III. (Series V.).—Table of Discharges per Foot Run of Submerged Falls.

$H = 2\frac{1}{2}.$

|  | Unit Increment, 12·68868 | Unit Increment, ·0007856 | Mean Velocities of Approach. |  |  |  |  |  |  |  |  |
|---|---|---|---|---|---|---|---|---|---|---|---|
|  |  |  | 0 | 1½ | 2 | 2½ | 3 | 3½ | 4 | 4½ | 5 |
|  |  |  | Multiplicands. |  |  |  |  |  |  |  |  |
| $d$ | $\sqrt{2gH}\times\frac{1}{3}(3d-H)$ | $c$ | — | 1·006 | 1·012 | 1·018 | 1·025 | 1·034 | 1·044 | 1·055 | 1·067 |
| 7½ | 84·590 | ·8095 | 68·458 | 68·87 | 69·28 | 69·69 | 70·17 | 70·79 | 71·47 | 72·22 | 73·05 |
| 8 | 90·935 | ·810 | 73·657 | 74·10 | 74·54 | 74·93 | 75·50 | 76·16 | 76·90 | 77·63 | 78·59 |
| 8½ | 97·279 | ·810 | 78·796 | — | 79·74 | 80·16 | 80·77 | 81·48 | 82·26 | 83·13 | 84·07 |
| 9 | 103·624 | ·811 | 84·039 | — | 85·05 | 85·49 | 86·14 | 86·90 | 87·74 | 88·66 | 89·67 |
| 9½ | 109·968 | ·811 | 89·104 | — | 90·25 | 90·74 | 91·42 | 92·22 | 93·11 | 94·09 | 95·16 |
| 10 | 116·312 | ·812 | 94·445 | — | 95·51 | 96·08 | 96·81 | 97·66 | 98·60 | 99·64 | 100·77 |
| 11 | 129·000 | ·813 | 104·748 | — | — | 106·56 | 107·17 | 108·31 | 109·36 | 110·51 | 111·77 |
| 12 | 141·690 | ·813 | 115·194 | — | — | 117·19 | 118·07 | 119·11 | 120·24 | 121·53 | 122·91 |
| 13 | 154·379 | ·814 | 125·665 | — | — | 128·05 | 128·81 | 129·94 | 131·19 | 132·58 | 134·19 |
| 14 | 167·067 | ·815 | 136·160 | — | — | 138·51 | 139·56 | 140·14 | 142·15 | 143·65 | 145·30 |
| 15 | 179·756 | ·816 | 146·681 | — | — | 149·22 | 150·00 | 151·67 | 153·13 | 154·75 | 156·51 |
| 16 | 192·445 | ·817 | 157·228 | — | — | — | 161·16 | 162·56 | 164·14 | 165·90 | 167·80 |
| 17 | 205·133 | ·817 | 167·594 | — | — | — | 171·80 | 173·30 | 174·97 | 176·81 | 178·82 |
| 18 | 217·822 | ·818 | 178·175 | — | — | — | 182·63 | 184·23 | 186·01 | 189·78 | 190·01 |
| 19 | 230·511 | ·819 | 188·789 | — | — | — | 193·51 | 195·21 | 197·10 | 199·16 | 200·05 |
| 20 | 243·199 | ·820 | 199·423 | — | — | — | 204·42 | 206·21 | 208·20 | 210·20 | 212·79 |
| 21 | 255·888 | ·820 | 209·818 | — | — | — | 215·07 | 216·96 | 219·06 | 221·40 | 223·90 |

N.B.—The multiplicand for $V = 6$ is 1·093.
„ „ „ $V = 7$ is 1·123.

## TABLE III. (SERIES VI.).—TABLE OF DISCHARGES PER FOOT RUN OF SUBMERGED FALLS.

$H = 3.$

| | Unit Increment, 13·8997 | Unit Increment, ·0006546 | MEAN VELOCITIES OF APPROACH. | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 0 | 1½ | 2 | 2½ | 3 | 3½ | 4 | 4½ | 5 |
| | | | MULTIPLICANDS. | | | | | | | | |
| $d$ | $\sqrt{2gH} \times \frac{1}{3}(3d - H)$ | $c$ | — | 1·005 | 1·010 | 1·014 | 1·021 | 1·029 | 1·037 | 1·046 | 1·059 |
| 9 | 111·198 | ·8095 | 89·991 | 90·45 | 90·89 | 91·21 | 91·88 | 92·60 | 93·32 | 94·13 | 95·03 |
| 9½ | 118·148 | ·810 | 95·700 | 96·18 | 96·66 | 97·04 | 97·71 | 98·47 | 99·24 | 100·10 | 101·37 |
| 10 | 125·098 | ·810 | 101·330 | 101·84 | 102·34 | 102·75 | 103·46 | 104·27 | 105·08 | 105·99 | 107·31 |
| 10½ | 132·047 | ·811 | 107·090 | — | 108·16 | 108·59 | 109·34 | 110·20 | 111·05 | 112·02 | 113·41 |
| 11 | 138·997 | ·811 | 112·726 | — | 113·85 | 114·30 | 115·10 | 116·00 | 116·90 | 117·80 | 119·45 |
| 11½ | 145·947 | ·812 | 118·503 | — | 119·69 | 120·16 | 120·99 | 121·94 | 122·89 | 123·95 | 125·50 |
| 12 | 152·897 | ·812 | 124·152 | — | 125·40 | 125·89 | 126·76 | 127·75 | 128·75 | 129·86 | 131·48 |
| 12½ | 159·847 | ·812 | 129·796 | — | 131·10 | 131·61 | 132·52 | 133·56 | 134·60 | 135·77 | 137·45 |
| 13 | 166·797 | ·812 | 135·440 | — | — | 137·34 | 138·28 | 139·37 | 140·45 | 141·67 | 143·43 |
| 13½ | 173·747 | ·813 | 141·256 | — | — | 143·23 | 144·22 | 145·39 | 146·48 | 147·75 | 149·59 |
| 14 | 180·696 | ·813 | 146·906 | — | — | 148·60 | 149·99 | 151·17 | 152·64 | 153·67 | 155·57 |
| 14½ | 187·646 | ·813 | 152·521 | — | — | 154·69 | 155·76 | 156·98 | 158·20 | 159·57 | 161·56 |
| 15 | 194·596 | ·814 | 158·401 | — | — | 160·62 | 161·73 | 163·00 | 164·26 | 165·69 | 167·75 |
| 16 | 208·496 | ·815 | 169·924 | — | — | — | 173·89 | 174·85 | 176·21 | 177·74 | 179·95 |
| 17 | 222·396 | ·815 | 181·253 | — | — | — | 185·06 | 186·94 | 187·96 | 189·59 | 191·95 |
| 18 | 236·295 | ·816 | 192·817 | — | — | — | 196·87 | 198·41 | 199·95 | 201·22 | 204·20 |
| 19 | 250·195 | ·816 | 204·160 | — | — | — | 208·45 | 210·08 | 211·71 | 213·55 | 216·29 |
| 20 | 264·295 | ·817 | 215·766 | — | — | — | 220·30 | 223·02 | 223·75 | 225·69 | 226·93 |
| 21 | 277·994 | ·819 | 227·121 | — | — | — | 231·89 | 233·71 | 235·52 | 237·51 | 240·52 |

N.B.—The multiplicand for $V = 6$ is 1·079.
" " " $V \times 7$ is 1·105.

## TABLE III. (SERIES VII.).—TABLE OF DISCHARGES PER FOOT RUN OF SUBMERGED FALLS.

$H = 3\frac{1}{2}.$

| | Unit Increment, 15·0143 | Unit Increment, ·00056112 | MEAN VELOCITIES OF APPROACH. | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 0 | 1½ | 2 | 2½ | 3 | 3½ | 4 | 4½ | 5 |
| | | | MULTIPLICANDS. | | | | | | | | |
| $d$ | $\sqrt{2gH} \times \frac{1}{3}(3d - H)$ | $c$ | — | 1·004 | 1.008 | 1.012 | 1·018 | 1·023 | 1·033 | 1·041 | 1·049 |
| 10½ | 104·125 | ·8095 | 113·402 | — | 114·20 | 114·76 | 115·44 | 116·01 | 117·14 | 118·05 | 118·96 |
| 11 | 147·632 | ·810 | 119·582 | — | 120·82 | 121·02 | 121·57 | 122 33 | 123·53 | 124·48 | 125·44 |
| 11½ | 155·139 | ·810 | 125·663 | — | 126·19 | 127·20 | 127·95 | 128·58 | 129·98 | 130·84 | 131·83 |
| 12 | 162·646 | ·811 | 131·906 | — | 132·96 | 133·55 | 134·28 | 134·94 | 136·26 | 137·33 | 138·37 |
| 12½ | 170·154 | ·811 | 137·995 | — | — | 139·65 | 140·48 | 141·17 | 142·55 | 143·98 | 144·76 |
| 13 | 177·660 | ·811 | 144·087 | — | — | 145·82 | 146·68 | 147·40 | 148·84 | 150·00 | 151·15 |
| 13½ | 185·168 | ·811 | 150·171 | — | — | 151·97 | 152·87 | 153·63 | 155·13 | 156·33 | 157·53 |
| 14 | 192·675 | ·812 | 156·452 | — | — | 158·33 | 159·27 | 160·05 | 161·62 | 162·87 | 164·12 |
| 14½ | 200·172 | ·812 | 162·548 | — | — | 164·50 | 165·47 | 166·29 | 167·91 | 169·21 | 170·51 |
| 15 | 207·689 | ·812 | 168·644 | — | — | 170·67 | 171·68 | 172·52 | 174·20 | 175·56 | 176·91 |
| 16 | 222·704 | ·813 | 181·059 | — | — | — | 184·31 | 185·22 | 187·03 | 188·48 | 189·93 |
| 17 | 237·718 | ·813 | 193·265 | — | — | — | 196·75 | 197·71 | 199·64 | 201·19 | 202·73 |
| 18 | 252·732 | ·814 | 205·724 | — | — | — | 209·43 | 210·46 | 212·51 | 214·06 | 215·80 |
| 19 | 267·747 | ·814 | 217·940 | — | — | — | 221·87 | 222·96 | 225·14 | 226·88 | 228 63 |
| 20 | 282·761 | ·815 | 230·450 | — | — | — | 234·60 | 235·75 | 238·05 | 239·89 | 241·74 |
| 21 | 297·776 | ·815 | 242·688 | — | — | — | 246·76 | 248·27 | 250·70 | 252·64 | 254·58 |

N.B.—The multiplicand for $V = 6$ is 1·068.
" " " $V = 7$ is 1·091.

## TABLE III. (SERIES VIII.).—TABLE OF DISCHARGES PER FOOT RUN OF SUBMERGED FALLS.

$H = 4.$

|  | Unit Increment, 16·05 | Unit Increment, ·00049. | MEAN VELOCITIES OF APPROACH. |  |  |  |  |  |  |  |  |
|---|---|---|---|---|---|---|---|---|---|---|---|
|  |  |  | 0 | 2 | 2½ | 3 | 3½ | 4 | 4½ | 5 | 6 |
|  |  |  | MULTIPLICANDS. |  |  |  |  |  |  |  |  |
| $d$ | $\sqrt{2gH} \times \frac{1}{3}(d - H)$ | $c$ | 000 | 1·007 | 1·011 | 1·016 | 1·022 | 1·028 | 1·035 | 1·043 | 1·060 |
| 12 | 171·200 | ·8094 | 138·425 | 139·44 | 140·00 | 140·68 | 141·44 | 142·33 | 143·27 | 144·38 | 146·80 |
| 13 | 187·25 | ·810 | 151·676 | 152·73 | 153·34 | 154·10 | 155·01 | 155·92 | 156·98 | 158·20 | 160·77 |
| 14 | 203·30 | ·810 | 164·673 | 165·83 | 166·49 | 167·31 | 168·30 | 169·28 | 170·44 | 171·75 | 174·55 |
| 15 | 219·35 | ·811 | 177·893 | — | 179·85 | 180·74 | 181·80 | 184·87 | 184·12 | 185·54 | 188·57 |
| 16 | 235·40 | ·811 | 190·909 | — | — | 193·96 | 195·11 | 196·26 | 197·59 | 199·12 | 203·36 |
| 17 | 251·40 | ·812 | 204·178 | — | — | 207·45 | 208·67 | 209·90 | 211·32 | 212·96 | 216·43 |
| 18 | 267·50 | ·812 | 217·210 | — | — | 220·68 | 221·99 | 223·29 | 224·81 | 226·55 | 230·24 |
| 19 | 283·55 | ·813 | 230·526 | — | — | 234·22 | 235·60 | 236·98 | 238·60 | 240·44 | 244·36 |
| 20 | 299·60 | ·813 | 243·575 | — | — | 247·47 | 248·93 | 250·40 | 252·10 | 254·05 | 258·19 |
| 21 | 315·65 | ·814 | 256·940 | — | — | 261·05 | 262·60 | 264·13 | 265·94 | 267·99 | 272·36 |

N.B.—The multiplicand for $V = 7$ is 1·080.

## SERIES IX.

$H = 4\frac{1}{2}.$

|  | Unit Increment, 17·0236 | Unit Increment, ·0004366 | MULTIPLICANDS. |  |  |  |  |  |  |  |  |
|---|---|---|---|---|---|---|---|---|---|---|---|
| $d$ | $\sqrt{2gH} \times \frac{1}{3}(3d - H)$ | $c$ | 000 | — | — | 1·015 | 1·020 | 1·025 | 1·030 | 1·039 | 1·057 |
| 13½ | 204·283 | ·8094 | 165·324 | — | — | 167·80 | 168·63 | 169·46 | 170·28 | 171·77 | 174·75 |

N.B.—The multiplicand for $V = 7$ is 1·072.

## SERIES X.

$H = 5.$

|  | Unit Increment, 17·9440 | Unit Increment, ·000385 | MULTIPLICANDS. |  |  |  |  |  |  |  |  |
|---|---|---|---|---|---|---|---|---|---|---|---|
| $d$ | $-2gH \times \frac{1}{3}(3d - H)$ | $c$ | — | — | — | — | 1·018 | 1·023 | 1·028 | 1·035 | 1·049 |
| 15 | 239·258 | ·8094 | 193·630 | — | — | — | 197·12 | 198·08 | 199·05 | 200·41 | 203·12 |

N.B.—The multiplicand for $V = 7$ is 1·065.

## TABLE III. (SUPPLEMENTARY).—FOR INTERMEDIATE VALUES OF $H$.

$H = \frac{3}{4}$.

|  | Unit Increment, 6·9496 | Unit Increment, ·00263 | MEAN VELOCITIES OF APPROACH. |  |  |  |  |  |  |  |  |  |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
|  |  |  | 0 | 1 | 1½ | 2 | 2½ | 3 | 3½ | 4 | 4½ | 5 |
|  |  |  | MULTIPLICANDS. |  |  |  |  |  |  |  |  |  |
| $d$ | $\sqrt{2gH}\times\frac{1}{3}(3d-H)$ | $C$ | 000 | 1·010 | 1·021 | 1·037 | 1·056 | 1·079 | 1·104 | 1·133 | 1·164 | 1·198 |
| 2¼ | 13·899 | ·8094 | 11·249 | 11·361 | 11·485 | 11·665 | 11·868 | 12·138 | 12·419 | 12·745 | 13·094 | 13·476 |

$H = 1\frac{1}{4}$.

|  | Unit Increment, 8·52337 | Unit Increment, ·001572 | MULTIPLICANDS. |  |  |  |  |  |  |  |  |  |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
|  |  |  |  | 1·007 | 1·013 | 1·022 | 1·035 | 1·050 | 1·066 | 1·084 | 1·104 | 1·125 |
| 3¾ | 29·906 | ·8094 | 24·204 | 24·373 | 24·519 | 24·760 | 25·051 | 25·414 | 25·801 | 26·237 | 26·721 | 27·229 |

$H = 1\frac{3}{4}$.

|  | Unit Increment, 10·61872 | Unit Increment, ·001123 | MULTIPLICANDS. |  |  |  |  |  |  |  |  |  |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
|  |  |  |  | 1·005 | 1·010 | 1·016 | 1·026 | 1·036 | 1·048 | 1·062 | 1·077 | 1·093 |
| 5¼ | 49·558 | ·8094 | 40·094 | — | 40·494 | 40·735 | 41·136 | 41·537 | 42·018 | 42·579 | 42·982 | 43·822 |

$H = 2\frac{1}{4}$.

|  | Unit Increment, 12·0375 | Unit Increment, ·000873 | MULTIPLICANDS. |  |  |  |  |  |  |  |  |  |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
|  |  |  | — | — | 1·008 | 1·013 | 1·020 | 1·028 | 1·038 | 1·046 | 1·050 | 1·073 |
| 6¾ | 72·225 | ·8094 | 58·451 | — | 58·919 | 59·211 | 59·621 | 60·088 | 60·672 | 61·183 | 61·958 | 62·718 |

(**22**) Now with regard to the effect of velocity of approach. Inspection of Table I., calculated from Formula (9), shows a progressive fall in the value of the multiplicands of the coefficient in inverse ratio to that of *d*. The same occurs in the multiplicands for a submerged fall if Formula (11) be used. These latter multiplicands are obtained by dividing the discharge found by Formula (11) by that by Formula (10). The application of Formula (9), substituting *H* for *d*, is not correct. Thus in Table VI., Series II., when $H = 1$ and $d = 4$, the discharge according to Formula (10), *i.e.*, when $V = 0$, is 29·425 cubic feet per foot run. The corresponding discharge for $V = 5$ shown in the last column and same line is 35·685 cubic feet. The multiplicand in this case will be $\frac{35{\cdot}685}{29{\cdot}425}$ or 1·212 nearly. Now the corresponding multiplicand for the same value of *d* for free overfalls is (*vide* Table I., last column) 1·118, and similarly in every other case the multiplicands of the coefficients for submerged falls calculated by Formula (11) are higher than those of Formula (8) adapted to free overfalls. As however with $H$ being a fixed value in the series, and the initial value of $d$ being $= 3H$, the discharge with $V = 0$ is, as we have seen, identical with that of a free overfall, $d$ having the same value in both cases. This being so, it naturally follows that the multiplicands, to allow for the increase of discharge due to velocity of approach, must likewise be the same as in Table I. Otherwise the discharges of the initial values of $d$ will not coincide, when calculated as a free overfall, or as a submerged fall, which is clearly essential. Consequently the multiplicands of the first line in the several series in Table III. are taken from Table I. Now as regards the higher values of $d$. The theory adopted is that with the increase of $d$ beyond the initial value the conditions gradually change from those of a free to those of a submerged overfall, when naturally Formula (11) would apply in its entirety with its higher multiplicand. As these multiplicands fall in value in inverse ratio of $d$, eventually a value of $d$ will be found such that the resulting multiplicand is so reduced, that it coincides with the lower initial multiplicand applicable to the initial value of $d$. This point or value of $d$, as can easily be tested from Table V., is found to fall beyond the practical extreme depth of film passing over a weir in rivers in flood, which is about 25 feet. This being the case, the solution of the difficulty is simplified by the adoption of the initial multiplicands right through without alteration, as has been done in Table III.

(**23**) The mean velocity of approach, $Va$, which we have been considering, is naturally some function of that of the normal river current. Now the discharge $Q = A V = A_1 Va$, in which expression $A_1$ is the area of the water section at or near the weir or obstruction. The value of $A_1$ may be more or may be less than that of $A$, or else, as with dwarf river weirs, it may be considered equal to it. In deep reservoirs which do not silt up to any great extent $Va$ is less than $V$. A proper appraisement of the value of $A_1$ is essential to the correct valuation of $Va$, and in estimating the area of $A_1$ the depth of film passing the weir crest must be assumed tentatively (*vide* paras. 15 and 33, Chap. III.).

(**24**) The coefficients deduced by Mr. Rhind from calculations of the discharge of Orissa River in flood, published in Vol. LXXXV., "Min. Pro. Inst. C.E." are, in the author's opinion, misleading for the following reasons. Firstly, the body of water is considered divided into two parts, the free and the submerged, and a different formula used for either portion. In both of these the velocity of approach is added to the observed head, as in Formula (6), which is used for the lower portion. This must give erroneous results, as no deduction is made for the discharge due to the increased head. Secondly, the coefficients used for the upper portion are those deduced from the ancient Metz experiments of 1832, which are now entirely superseded by the Lowell coefficient of ·623, and even if the latter were used it is quite problematical whether they would apply to the free part of submerged overfall, the conditions of which are certainly different to those of the same depth of film passing free over the crest of a solid weir. Thirdly, the value of the coefficients is deduced from the calculated discharge of a large river in flood, which may easily be in error to the extent of 15 or 20 per cent. Fourthly, the average or mean coefficient deduced, the data of which are incorrectly quoted in "Irrigation in India," p. 305, are of no use till it is known to what formula it is applicable. A coefficient is always a multiplicand of some theoretical formula, but these so-called mean coefficients apply to two different formulas, and so are not applicable to Formulas (10) or (11), which should have been used in this case. No doubt if the whole calculations were gone through again with the use of the Formulas (10) or (11), the resulting coefficient could be obtained, but considering that the correct estimation of the actual discharge of the rivers is a matter of doubt, it would probably give only misleading results. A correct estimation of coefficients can only be made by experiments on a canal, the discharge of which can be calculated with precision.

**(25)** Another point to be noted is that the mean velocity of approach of a large wide river varies in different parts of the water section, and in such cases the rivers and weirs should be divided into parts, each with its own velocity of approach, and the multiplicand varied accordingly. The reduction of observed surface to mean velocities is treated of in par. 40. This has not been attempted in Mr. Rhind's computations, one surface velocity being common to all parts of the water section, which would certainly not be the actual case.

**(26)** In the above treatment of weirs we have dealt with open weirs, presumably across rivers; in canals, however, such weirs are not now designed, the great superiority of the notch type of weirs, which enables the water in the upper reach to be held up to normal level at whatever depth it may be, rendering its adoption universal for all canal falls.

In a notch fall the triangular, or rather trapezoidal, openings are built with a converging and diverging approach, similar to the so-termed adjutage attached to an orifice. This method of construction greatly increases the value of the coefficient of discharge through the notch, so that when thus constructed the modification due to end contraction, viz., $(l - \cdot 1nd)$, par. 11, need not be used, as it would have to be were the sides of the notch left square; instead of which the coefficient is slightly reduced. In "Irrigation in India" the coefficient employed with the very elaborately designed notches said to be used on the Chenab Canal is stated to be ·6, and in the examples given the same coefficient is used for submerged as for free overfalls. As previously explained in par. 7, the author has no hesitation in stating that this must be in error, as the coefficient for a submerged fall or orifice is invariably higher than that for a free opening. As most canal falls are submerged it is deemed that the coefficient should be reduced for a free overfall to about ·56, thus bringing the formula for free discharge per foot width to the following

$$Q = \cdot 3d\sqrt{d} \text{ or } = \cdot 3d^{\frac{3}{2}} \qquad (13)$$

(**27**) On the principle already explained, when the fall is two-thirds submerged, the coefficient applicable to Formula (10) is obtained by equating Formulas (10) and (12), and this initial coefficient amounts to ·7285 nearly. Tables IV. and V. have been prepared on these data, the first for free overfalls up to $d = 3H$, after which Table V. comes into operation.

### TABLE IV.—NOTCH FALLS.—TABLE OF DISCHARGES PER FOOT RUN. FREE OVERFALL.

| | MEAN VELOCITIES OF APPROACH. | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $d$ | $\cdot 3d^{\frac{3}{2}}$<br>0 | 1 | 1½ | 2 | 2½ | 3 | 3½ | 4 |
| ½ | 1·061 | 1·104 | 1·158 | 1·218 | | | | |
| 1 | 3·000 | 3·063 | 3·141 | 3·234 | 3·357 | | | |
| 1½ | 5·512 | 5·589 | 5·671 | 5·809 | 5·963 | 6·140 | | |
| 2 | 8·485 | 8·579 | 8·743 | 8·825 | 9·003 | 9·240 | | |
| 2½ | 11·858 | 11·965 | 12·084 | 12·250 | 12·463 | 12·712 | 12·985 | |
| 3 | 15·588 | 15·713 | 15·853 | 16·056 | 16 259 | 16·540 | 16·851 | |
| 3½ | 19·644 | 19·777 | 19·914 | 20·236 | 20·366 | 20·660 | 21·014 | |
| 4 | 24·000 | 24·144 | 24·288 | 24·505 | 24 792 | 25·128 | 25·488 | |
| 4½ | 28·638 | 28·781 | 28·953 | 29·182 | 29·469 | 29·812 | 30·213 | |
| 5 | 33·541 | — | 34·876 | 34·033 | 34·413 | 34·782 | 35·218 | 35·721 |
| 5½ | 38·696 | — | 39·083 | 39·277 | 39·663 | 40·012 | 40·476 | 40·979 |
| 6 | 44·091 | — | 44·487 | 44·672 | 45·103 | 45·458 | 45·943 | 46·516 |
| 6½ | 49·716 | — | 50·113 | 50·411 | 50·759 | 51·039 | 51·654 | 52·201 |
| 7 | 55·561 | — | — | 56·283 | 56·617 | 57·061 | 57·616 | 58·228 |
| 7½ | 61·619 | — | — | 62·359 | 62·728 | 63·160 | 63·714 | 64·330 |
| 8 | 67·882 | — | — | 68·697 | 68·969 | 69·444 | 70·055 | 70·734 |
| 8½ | 74·345 | — | — | 75·163 | 75·460 | 75·981 | 76·575 | 77·244 |
| 9 | 81·000 | — | — | 81·810 | 82·134 | 82·701 | 83·349 | 83·997 |
| 9½ | 87·843 | — | — | — | 89·073 | 89·600 | 90·215 | 90·918 |
| 10 | 94·869 | — | — | — | 96·102 | 96·671 | 97·240 | 98·094 |

### TABLE V.—NOTCH FALLS.—TABLE OF DISCHARGES PER FOOT RUN. SUBMERGED FALLS.

$H = 2.$

| | | | MEAN VELOCITIES OF APPROACH. | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | 0 | 1½ | 2 | 2½ | 3 | 3½ | 4 |
| | | Unit Increment, ·003755 | MULTIPLICANDS. | | | | | | |
| $d$ | $\sqrt{2gH} \times \frac{1}{3}(3d - H)$ | $c$ | 0 | 1·009 | 1·014 | 1·023 | 1·031 | 1·042 | 1·055 |
| 6 | 60·528 | ·7285 | 44·091 | 44·49 | 44·67 | 45·10 | 45·46 | 45·94 | 46·52 |
| 6½ | 66·203 | ·730 | 48·328 | 48·76 | 49·00 | 49·44 | 49·83 | 50·36 | 50·99 |
| 7 | 71·877 | ·732 | 52·614 | 53·09 | 53·35 | 53·82 | 54·25 | 54·82 | 55·51 |
| 7½ | 77·552 | ·734 | 56·923 | 57·17 | 57·69 | 58·23 | 58·69 | 59·31 | 60·05 |
| 8 | 83·226 | ·736 | 61·254 | 61·81 | 62·11 | 62·66 | 63·15 | 63·83 | 64·62 |
| 8½ | 88·901 | ·738 | 65·609 | 66·20 | 66·53 | 67·12 | 67·64 | 68·36 | 69·23 |
| 9 | 94·575 | ·740 | 69·985 | — | 70·97 | 71·59 | 72·15 | 72·93 | 73·84 |
| 9½ | 100·249 | ·742 | 74·385 | — | 75·43 | 76·10 | 76·69 | 77·51 | 78·47 |
| 10 | 105·924 | ·744 | 78·804 | — | 79·91 | 80·62 | 81·25 | 82·12 | 83·14 |

### TABLE V.—NOTCH FALLS.—TABLE OF DISCHARGES, ETC.—continued.

$H = 2\frac{1}{2}$.

|  |  | Value of Coefficient. | Mean Velocities of Approach. |  |  |  |  |  |  |
|---|---|---|---|---|---|---|---|---|---|
|  |  |  | 0 | 1½ | 2 | 2½ | 3 | 3½ | 4 |
|  |  |  | Multiplicands. |  |  |  |  |  |  |
| d | $\sqrt{2gH}\sqrt{(d-H)}$ | c | 0 | 1·006 | 1·012 | 1·018 | 1·025 | 1·034 | 1·044 |
| 7½ | [illegible] | ·7285 | [illegible] | [illegible] | [illegible] | [illegible] | [illegible] | [illegible] | [illegible] |
| 8 | [illegible] | ·729 | [illegible] | [illegible] | [illegible] | [illegible] | [illegible] | [illegible] | [illegible] |
| 8½ | [illegible] | ·730 | [illegible] | 71·44 | [illegible] | [illegible] | 72·79 | 73·43 | [illegible] |
| 9 | [illegible] | ·730 | [illegible] | — | [illegible] | [illegible] | [illegible] | [illegible] | [illegible] |
| 9½ | [illegible] | ·731 | [illegible] | — | [illegible] | [illegible] | 82·40 | [illegible] | [illegible] |
| 10 | [illegible] | ·731 | 85·024 | — | [illegible] | 86·55 | 87·15 | [illegible] | [illegible] |

$H = 3$.

|  |  |  | Multiplicands. |  |  |  |  |  |  |
|---|---|---|---|---|---|---|---|---|---|
|  |  |  | 0 | 1·005 | 1·010 | 1·014 | 1·021 | 1·029 | 1·037 |
| 9 | [illegible] | ·7285 | 81·000 | — | 81·81 | 82·13 | 82·70 | [illegible] | 84·00 |
| 9½ | [illegible] | ·729 | 86·130 | — | [illegible] | 87·34 | 87·94 | [illegible] | [illegible] |
| 10 | [illegible] | ·729 | 91·196 | — | 92·11 | 92·47 | 93·11 | 93·54 | 94·57 |

$H = 3\frac{1}{2}$.

|  |  |  | Multiplicands. |  |  |  |  |  |  |
|---|---|---|---|---|---|---|---|---|---|
|  |  |  |  | — | 1·009 | 1·012 | 1·018 | 1·023 | 1·033 |
| 10½ | [illegible] | ·7285 | 102·072 | — | 102·99 | 103·30 | 103·91 | 104·43 | 105·85 |

### TABLE VI.—THEORETICAL DISCHARGES OF SUBMERGED WEIRS FROM FORMULA (11). $\sqrt{2g}\left[(d + h)\sqrt{H + h} - \frac{1}{3}\left\{(H + h)^{\frac{3}{2}} + 2h^{\frac{3}{2}})\right\}\right]$

$v = \cdot0155\, V_2$. ($c$ and $l$ = unity.)

$H = \frac{1}{2}$.

|  | Mean Velocities of Approach. |  |  |  |  |  |  |  |  |  |
|---|---|---|---|---|---|---|---|---|---|---|
| d | $\sqrt{2gH}\sqrt{(d-H)}$ 0 | 1 | 1½ | 2 | 2½ | 3 | 3½ | 4 | 4½ | 5 |
| 1½ | 7·566 | 7·727 | 7·923 | 8·183 | 8·528 |  |  |  |  |  |
| 2 | 10·403 | 10·606 | 10·856 | 11·190 | 11·634 | 12·115 |  |  |  |  |
| 2½ | 13·241 | 13·486 | 13·789 | 14·197 | 14·720 | 15·317 |  |  |  |  |
| 3 | 16·075 | 16·365 | 16·722 | 17·204 | 17·817 | 18·519 | 19·273 | 20·123 | 21·077 | 22·072 |
| 3½ | 18·912 | 19·245 | 19·655 | 20·211 | 20·913 | 21·721 | 22·604 | 23·592 | 24·695 | 25·851 |
| 4 | 21·752 | 22·124 | 22·588 | 23·218 | 24·010 | 24·923 | 25·935 | 27·061 | 28·313 | 29·629 |
| 4½ | 24·590 | 25·003 | 25·521 | 26·225 | 27·106 | 28·125 | 29·266 | 30·530 | 31·932 | 33·408 |
| 5 | 27·427 | 27·883 | 28·454 | 29·232 | 30·303 | 31·326 | 32·597 | 33·998 | 35·550 | 37·186 |
| 5½ | 30·264 | — | 31·387 | 32·238 | 33·300 | 34·528 | 35·928 | 37·467 | 39·168 | 40·965 |
| 6 | 33·101 | — | 34·320 | 35·245 | 36·395 | 37·730 | 39·259 | 40·936 | 42·786 | 44·743 |
| 6½ | 35·938 | — | — | 38·252 | 39·491 | 40·932 | 42·590 | 44·405 | 46·404 | 48·521 |
| 7 | 38·776 | — | — | 41·259 | 42·588 | 44·134 | 45·930 | 47·874 | 50·022 | 52·300 |
| 7½ | 41·613 | — | — | 44·266 | 45·684 | 47·335 | 49·251 | 51·342 | 53·640 | 56·078 |
| 8 | 44·456 | — | — | 47·273 | 48·781 | 50·537 | 52·582 | 54·811 | 57·258 | 59·857 |
| 8½ | 47·288 | — | — | — | — | 53·739 | 55·913 | 58·280 | 60·876 | 63·635 |
| 9 | 50·124 | — | — | — | — | 56·941 | 59·244 | 61·749 | 64·494 | 67·414 |
| 9½ | 52·962 | — | — | — | — | 60·143 | 63·575 | 65·218 | 68·113 | 71·192 |
| 10 | 55·800 | — | — | — | — | 63·344 | 65·906 | 68·686 | 71·731 | 74·970 |

## TABLE VI.—THEORETICAL DISCHARGES, ETC.—*continued.*

### $H = 1$.

| | | MEAN VELOCITIES OF APPROACH. | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| $d$ | $\sqrt{2gH} \times \frac{1}{3}(3d-H)$<br>0 | 1 | 1½ | 2 | 2½ | 3 | 3½ | 4 | 4½ | 5 |
| 3 | 21·400 | 21·622 | 21·912 | 22·297 | 22·761 | 23·339 | 23·997 | 24·711 | 25·497 | 26·348 |
| 3½ | 25·412 | 25·669 | 25·992 | 26·430 | 26·962 | 27·619 | 28·372 | 29·191 | 30·102 | 31·016 |
| 4 | 29·425 | 29·710 | 30·071 | 30·563 | 31·162 | 31·898 | 32·746 | 33·271 | 34·707 | 35·685 |
| 4½ | 33·437 | 33·751 | 34·151 | 34·696 | 35·363 | 36·178 | 37·121 | 38·151 | 39·313 | 40·354 |
| 5 | 37·450 | 37·792 | 38·230 | 38·829 | 39·563 | 40·457 | 41·495 | 42·632 | 43·918 | 45·022 |
| 5½ | 41·462 | 41·834 | 42·310 | 42·962 | 43·763 | 44·736 | 45·869 | 47·112 | 48·524 | 49·691 |
| 6 | 45·475 | 45·875 | 46·390 | 47·095 | 47·963 | 49·016 | 50·244 | 51·592 | 53·129 | 54·360 |
| 6½ | 49·487 | 49·916 | 50·469 | 51·228 | 52·164 | 53·295 | 54·618 | 56·072 | 57·734 | 59·028 |
| 7 | 53·500 | 53·597 | 54·549 | 55·361 | 56·365 | 57·575 | 58·993 | 60·553 | 62·349 | 63·697 |
| 7½ | 57·512 | 57·999 | 58·629 | 59·494 | 60·565 | 61·854 | 63·367 | 65·033 | 66·945 | 68·366 |
| 8 | 61·525 | 62·040 | 62·708 | 63·627 | 64·765 | 66·133 | 67·741 | 69·514 | 71·550 | 73·035 |
| 8½ | 65·554 | — | — | — | — | 70·413 | 72·116 | 73·994 | 76·156 | 77·703 |
| 9 | 69·550 | — | — | — | — | 74·692 | 76·490 | 78·474 | 80·761 | 82·372 |
| 9½ | 73·562 | — | — | — | — | 78·971 | 80·864 | 82·954 | 85·367 | 87·041 |
| 10 | 77·575 | — | — | — | — | 83·251 | 85·239 | 87·434 | 89·972 | 91·709 |

### $H = 1\frac{1}{2}$.

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 4½ | 39·315 | — | 39·752 | 40·428 | 41·023 | 41·746 | 42·583 | 43·509 | 44·529 | 45·633 |
| 5 | 44·229 | — | 44·650 | 45·440 | 46·092 | 46·880 | 47·796 | 48·811 | 49·929 | 51·143 |
| 5½ | 49·143 | — | 49·549 | 50·452 | 51·161 | 52·015 | 53·010 | 54·113 | 55·330 | 56·652 |
| 6 | 54·057 | — | 54·037 | 55·464 | 56·231 | 57·149 | 58·223 | 59·415 | 60·731 | 62·161 |
| 6½ | 58·972 | — | 59·346 | 60·476 | 61·300 | 62·283 | 63·437 | 64·718 | 66·131 | 67·670 |
| 7 | 63·886 | — | 64·244 | 65·488 | 66·370 | 67·417 | 68·650 | 70·020 | 71·532 | 73·179 |
| 7½ | 68·800 | — | 69·142 | 70·501 | 71·439 | 72·552 | 73·863 | 75·322 | 76·933 | 78·689 |
| 8 | 73·714 | — | 74·041 | 75·513 | 76·508 | 77·686 | 79·077 | 80·624 | 82·323 | 84·198 |
| 8½ | 78·629 | — | — | 80·525 | 81·578 | 82·820 | 84·290 | 85·226 | 87·734 | 89·707 |
| 9 | 83·543 | — | — | 85·537 | 86·648 | 87·954 | 89·504 | 91·228 | 93·134 | 95·217 |
| 9½ | 88·457 | — | — | — | 91·717 | 93·089 | 94·717 | 96·530 | 98·535 | 100·726 |
| 10 | 93·372 | — | — | — | 96·786 | 98·223 | 99·931 | 101·832 | 103·936 | 106·235 |

### $H = 2$.

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 6 | 60·528 | 61·251 | 61·820 | 62·534 | 63·370 | 64·361 | 65·461 | 66·681 | 67·990 |
| 6½ | 66·202 | 66·971 | 67·580 | 68·342 | 69·235 | 70·295 | 71·473 | 72·781 | 74·181 |
| 7 | 71·877 | 72·692 | 73·339 | 74·149 | 75·100 | 76·329 | 77·485 | 78·881 | 80·373 |
| 7½ | 77·551 | 78·413 | 79·098 | 79·956 | 80·965 | 82·163 | 83·497 | 84·981 | 86·165 |
| 8 | 83·226 | 84·133 | 84·857 | 85·763 | 86·830 | 88·097 | 89·509 | 91·081 | 92·757 |
| 8½ | 88·900 | — | 90·617 | 91·571 | 92·696 | 94·032 | 95·521 | 97·181 | 98·948 |
| 9 | 94·575 | — | 96·376 | 97·378 | 98·561 | 99·966 | 101·533 | 103·280 | 105·140 |
| 9½ | 100·249 | — | 102·135 | 103·185 | 104·426 | 105·900 | 107·545 | 109·380 | 111·332 |
| 10 | 105·924 | — | 107·894 | 108·992 | 110·291 | 111·834 | 113·557 | 115·480 | 117·524 |

### $H = 2\frac{1}{2}$.

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 7½ | 84·590 | — | 86·039 | 86·846 | 87·792 | 88·949 | 90·177 | 91·573 | 93·095 |
| 8 | 90·935 | — | 92·458 | 93·310 | 94·307 | 95·527 | 96·825 | 98·301 | 99·910 |
| 8½ | 97·779 | — | 98·876 | 99·773 | 100·822 | 102·104 | 103·473 | 105·029 | 106·725 |
| 9 | 103·623 | — | 105·295 | 106·237 | 107·337 | 108·683 | 110·121 | 111·756 | 113·549 |
| 9½ | 109·968 | — | 111·713 | 112·700 | 113·852 | 115·261 | 116·769 | 118·484 | 120·355 |
| 10 | 116·312 | — | 118·132 | 119·163 | 120·367 | 121·839 | 123·418 | 125·211 | 127·171 |

TABLE VI.—THEORETICAL DISCHARGES, ETC.—*continued.*

$H = 3.$

| | | Mean Velocities of Approach. | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| $d$ | $\sqrt{2gH}$<br>$\frac{1}{3}(3d-H)$. | $1\frac{1}{2}$ | 2 | $2\frac{1}{2}$ | 3 | $3\frac{1}{2}$ | 4 | $4\frac{1}{2}$ | 5 |
| 9 | 111·198 | — | 112·775 | 113·668 | 114·719 | 115·969 | 117·367 | 118·942 | 120·618 |
| $9\frac{1}{2}$ | 118·147 | — | 119·793 | 120·727 | 121·824 | 123·131 | 124·594 | 126·252 | 127·999 |
| 10 | 125·097 | — | 126·811 | 127·784 | 128·929 | 130·293 | 131·821 | 133·563 | 135·379 |
| | | | | | $H = 3\frac{1}{2}.$ | | | | |
| $10\frac{1}{2}$ | 140·124 | — | 141·829 | 142·796 | 143·987 | 145·306 | 146·826 | 148·532 | 151·665 |
| 11 | 147·632 | — | 149·398 | 150·401 | 151·686 | 153·009 | 154·589 | 156·364 | 159·663 |
| 12 | 155·139 | — | 156·968 | 158·007 | 159·385 | 160·712 | 162·352 | 164·196 | 167·661 |
| $12\frac{1}{2}$ | 162·646 | — | 164·537 | 165·613 | 167·084 | 168·416 | 170·115 | 172·028 | 175·659 |
| | | | | | $H = 4.$ | | | | |
| 12 | 171·200 | — | 173·011 | 174·102 | 175·305 | 176·654 | 178·402 | 180·258 | 182·251 |
| $12\frac{1}{2}$ | 179·225 | — | 181·094 | 182·225 | 183·466 | 184·837 | 186·667 | 188·567 | 190·651 |
| 13 | 187·250 | — | 189·176 | 190·348 | 191·527 | 193·020 | 194·933 | 196·896 | 199·051 |

**(28)** The use of Tables II. to V. will now be further illustrated.

Table II. Free overfall.

1. For values of *d* intermediate to those given in the Table. The discharges can be obtained roughly by interpolation vertically, but as the increase is proportional to $d^{\frac{3}{2}}$ and not to *d*, it would be more accurate to calculate the discharge with $V = 0$ from Formula (8), viz. $3·333d^{\frac{3}{2}}$. In cases where *d* exceeds the limit in the Table, calculation of both the discharge when $V = 0$ under Formula (8) as well as the multiplicand by Formula (9) will have to be made.

2. For values of *V* intermediate between those given in the Table the discharge, if *d* is in the Table, can very well be obtained by interpolation, as the differences between the vertical columns in the Table are not great. For values of *V* above 7 feet per second the multiplicand will have to be calculated from Formula (9), but such high mean velocities are rare.

**(29)** Table III. Submerged fall.

For values of *d* intermediate to those given in the Table (*V* and *H* remaining the same), the discharge can be obtained with very small sacrifice of accuracy by interpolation.

When *d* exceeds 21 feet, the limit of the Table, the corresponding value of Formula (10) in the second column can easily be obtained by adding on the constant unit the requisite number of times to the last entry or $(d - 21)i + m$ in which *m* is the discharge in column 2 at 21 feet, *i* the constant, and *d* the depth of film higher than 21 feet. Having obtained the second column entry, the third column entry, or the coefficient, can be obtained in like manner. The product of these two gives the discharge in column 4 when $V = o$, and that for any integral value of $V$ can be obtained at once by using proper multiplicands.

For intermediate values of $V$, the entry in column 4, when $V = 0$, has to be multiplied by the proper multiplicand, the value of which may be obtained by interpolation.

The following example will illustrate this:—

Let $d = 8{\cdot}4$ and $V = 3{\cdot}3$, $H$ being $\frac{1}{2}$. The discharge in column 4 ($V = 0$) (Table III., Series I.) will be as follows:—

The difference between discharge for $d = 8$ and $d = 8\frac{1}{2}$ will be $39{\cdot}580 - 37{\cdot}115 = 2{\cdot}465$, and the increment to be added to the lower expression will be $2{\cdot}465 \times \frac{{\cdot}4}{{\cdot}5} = 1{\cdot}972$, and the discharge for column 4, $37{\cdot}115 + 1{\cdot}972 = 39{\cdot}087$.

Again, the given value for $V$ lies intermediate between $V = 3$ and $V = 3\frac{1}{2}$, whose multiplicands are 1·114 and 1·151. Their difference being ·037 and the addendum to the lower value will be ${\cdot}037 + \frac{{\cdot}3}{{\cdot}5} = {\cdot}022$, and hence the required multiplicand will be $1{\cdot}114 + {\cdot}022 = 1{\cdot}136$. The discharge required will then be $39{\cdot}087 \times 1{\cdot}136 = 44{\cdot}403$ cubic feet.

**(30)** When $H$ varies, the difference in column 4 is so great in the lower values of $\frac{1}{2}$, 1 and $1\frac{1}{2}$ that the supplementary skeleton tables have been supplied for $H = \frac{3}{4}$, $1\frac{1}{4}$, $1\frac{3}{4}$ and $2\frac{1}{4}$, the initial values of columns 1 to 3 being given with the constants and the multiplicands, which information greatly facilitates the calculation of any discharges. If any special value of $H$ not included in the Tables is really required, the procedure is as follows:—

(1) The item in column 2 is worked out by Formula (10).

(2) The initial coefficient is ·8095 when $d = 3H$, the unit increment will be $\frac{1 - c^{\circ}}{97H} = \frac{1916}{97H} = i$, and the co-efficient due to $d$ will be ${\cdot}8095 + (d - d^{\circ})\ i$.

(3) The product of columns 1 and 2, give the items in column 3, which is the discharge when $V = 0$.

(4) The proper multiplicand is obtainable by interpolation from Table I., with $d = 3H$ or calculated from Formula (7).

Example, let $d = 6$, $H = 1\frac{1}{4}$, $V = 2\frac{1}{2}$, then initial $d$ or $d^{\circ}$ will be $3H$ or $3{\cdot}75$. With $d = 6$ and $H = 1{\cdot}25$ the item in column 2 will be $8{\cdot}025 \times \sqrt{1{\cdot}25} \times \frac{1}{3}(18 - 1{\cdot}25) = 9{\cdot}4695 \times 5{\cdot}583 = 52{\cdot}868$. Further, the constant ($i$) to the co-efficient will be $\frac{{\cdot}1916}{97 \times 1\frac{1}{4}} = \frac{{\cdot}1916}{121{\cdot}25} = {\cdot}00158$. The addendum to initial co-efficient will be $(d - d^{\circ})\ i$ or $= (6 - 3{\cdot}75)\ i = 2{\cdot}25 \times {\cdot}0016 = {\cdot}0034$ and the proper coefficient in column 3, ${\cdot}8095 + {\cdot}0034 = {\cdot}813$.

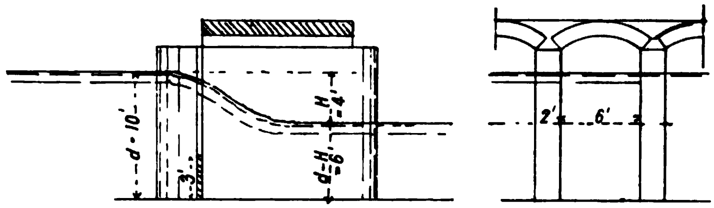
The discharge for $V = 0$ in column 4 will be $52 \times {\cdot}868 \times {\cdot}813 = 42{\cdot}982$. The multiplicand for $V = 2\frac{1}{2}$ and $d = 3H = 3{\cdot}75$ is obtained from Table I. by interpolation as 1·050 and the discharge required $= 42{\cdot}982 \times 1{\cdot}050 = 45.13$ cubic feet per foot run.

For higher values of $H$ above $2\frac{1}{2}$ it will be sufficiently near to obtain the discharge from column 4, by interpolation between the next lower and higher values of $H$, and the multiplicand due to $d = 3H$, by interpolation from Table I., and the product will be the approximately correct discharge.

(**31**) Tables IV. and V. are similar to Tables II. and III., and include all probable values of $d - H$ and of $V$. $H = 2$ is the lowest value, being for a 2 feet drop in the canal bed, which is about the least drop it would be worth building a fall for. The highest submerged value of $H$ will be $3\frac{1}{3}$. It is rare to find canals with water over 10 feet in depth.

(**32**) We will now proceed to examine the case of a fall through the openings of a Head regulating bridge, Escape head, or Undersluices.

Figs. 5 and 5a are views of a work of this description; the surface of water being free, and $H$ being over $\frac{1}{3}d$, the case apparently resolves itself into the simple one of free overfall, to which Formula (8) is applicable. The conditions, however, present some differences to those of a fall over a weir wall. The presence of the bottom floor diminishes the friction at the base of the current, and consequently in such cases as noticed in par. 9 the coefficient always rises in value. Against this rise is the diminution in effective width, due to the piers. As seen in par. 9, the reduction in the effective length of the water section would be $1nd$ or $\cdot 2 \times 10 = 2$ feet. This, however, applies only to the square and raised flanks of an ordinary weir wall, and not to piers provided with well shaped cutwaters, as should invariably be adopted. In the latter case the effect of the cutwaters is to nullify in a great measure the reduction alluded to.



(5a)

FIGS. 5, 5a.

In the absence of definite data on this subject the reduction in effective width may be taken as a quarter of that given in the formula, which would then stand at $l_1 = l - \cdot 1n\frac{d}{4}$, whence $l_1 = 6 - \cdot 5 = 5\cdot 5$.

(**33**) With regard to the coefficient, the ordinary one of ·623 would clearly be inapplicable. In the case of orifices, proper reference to the list in par. 8 shows that the rise of the coefficient in the case of bottom sluices is roughly about 2 points, *i.e.*, from about ·7 to ·9. This would apply to the undercurrent portion of the fall only, so that it is considered that the coefficient should be raised from ·623 to ·66 (the Castel coefficient) in the case of overfalls not considered as submerged, *i.e.*, when $\frac{H}{d}$ is below one-third. The discharge in such conditions would then be $Q = {}^{\cdot}66 \times 5{\cdot}35 d\sqrt{d}$ or $3{\cdot}53\, d^{\frac{3}{2}} \times \left(l - {}^{\cdot}2\frac{d}{4}\right)$ (14), or in the case under review $3{\cdot}53 \times 10 \times 3{\cdot}16 \times 5{\cdot}5 = 613{\cdot}5$ cubic feet per second. This is exclusive of velocity, of approach, to allow for which proper multiplicands given in Table I. should be applied.

**(34)** In case the fall was submerged above $\frac{d}{3}$, Table III. should be made use of, the initial coefficient in column 3, first line, should be raised by being multiplied by the fraction $\frac{66}{623}$ and would become $\cdot8095 \times 1{\cdot}0594 = \cdot8575$.

For higher values of $d$ it would probably be sufficiently near to adopt the same increment as given in the Table, or a new constant can be calculated by using the formula $\frac{1 - c^{\circ}}{97H}$ (par. 21) and the new coefficient for any value of $d$ required obtained by multiplying the increment by $d—d^{\circ}$ and adding to the initial coefficient or $c^{\circ}$, viz., ·8575. The coefficient ·66 is what is used by Jackson in the "Hydraulic Manual" for ordinary falls, but is now superseded by the Lowell Mills coefficient of ·623.

**(35)** If the arch were depressed below the surface of the water up stream, as is often the case in Head and Undersluices, the surface would not be free, and contraction occurring at the top as well as the sides of the opening would cause the latter to be classified as an orifice, when Formulas (2a) or (3) would be applicable, with a suitable coefficient selected from the list given in par. 8. In this case ·90 would probably apply.

**(36)** If a bottom gate or baulk were left in the grooves in Fig. 5 the discharge would be considerably modified. Supposing this obstruction to be 3 feet deep, then on the assumption that $H$ is of the same value, viz., 4 feet, $d$ will be reduced to 7 feet, and the coefficient falls back to ·623; the discharge according to Table 10 for a depth of 7 feet would then be

$$61{\cdot}728 \times 6 - ({}^{\cdot}2 \times \tfrac{7}{4}) = 61{\cdot}728 \times 5{\cdot}79 = 407{\cdot}5 \text{ cubic feet.}$$

As a matter of fact, $H$ would rise till equilibrium was produced somehow, a matter very difficult to estimate with any precision unless the whole river supply went through the sluices.

The depth of the escape channel, *i.e.*, of the tail water or $d — H$, depends on the slope of the channel and on its sectional area, *i.e.*, its discharging capacity, and has been dealt with in another place.

### *Discharge of Channels.*

**(37)** The formula for discharge of channels is

$$Q = AV = 100\, Ac\sqrt{RS} \qquad (15)$$

in which $R$ is the hydraulic mean depth, or $\frac{\text{area of section}}{\text{wetted perimeter}}$, and $S$ is the sine of the slope of water surface which can be assumed as parallel to the average slope of the bed of channel. The coefficient $c$ is obtainable from Kutter's formula, which has superseded all previous ones, as Bazin's. This formula expressed in feet is $c = \frac{\sqrt{R}}{100n} \times \frac{(m + 1\cdot811)}{(m + \sqrt{R})}$ (16), in which $m = n\left(41\cdot6 + \frac{\cdot00281}{S}\right)$ and $n$, termed the coefficient of rugosity, varies with the nature of the channel and its degree of smoothness. The values of $n$ for earthen channels are as follows:—

| | |
|---|---|
| For firm trimmed soil . . . | $n = \cdot020$ |
| For canals in good condition . . | $n = \cdot0225$ |
| For canals in average condition . | $n = \cdot025$ |
| For canals below average condition | $n = \cdot0275$ |
| For canals in defective condition, or rivers . . . . . . . | $n = \cdot030$ |
| For very defective channels . . | $n = \cdot035$ |

In calculations for canal discharges, $n$ is given a value either of ·025 or ·0225, the latter being used in the new Panjab Canals.

(**38**) This formula (16) being extremely cumbersome to work out, tables are invariably made use of in its application. Jackson's "Hydraulic Manual" (Lockwood) and the Canal and Culvert tables by the same author, are invaluable aids for calculation, as the tables provided enable rapid computation to be effected. First the value of the expression $100\sqrt{RS}$ for various values of $R$ and $S$ is obtained from one table and then the coefficient $c$ for varying values of $R$, $S$ and of $n$. These two values thus obtained, multiplied together and by the mean area $A = Q$, *i.e.*, the discharge in cubic feet per second. It is often convenient to substitute for $S$, the sine of the inclination of the water surface, $S$ in 1,000, termed $S^\circ$. The formula for velocity can then be expressed as $V = c\sqrt{10RS^\circ}$.

Thus if $R = 4$ and $S^\circ$ is ·2 per 1,000 $v$ will equal $2c\ \sqrt{2} = 2\cdot83c$ cubic feet per second.

For channels other than earth the values of $n$ are as follows:—

| | |
|---|---|
| Well planed planks . . . . . . | $n = \cdot009$ |
| Cement, plaster or enamelled pipes . . | $n = \cdot010$ |
| Unplaned timber . . . . . . . | $n = \cdot012$ |
| Ashlar and brickwork, cast and wrought iron . | $n = \cdot013$ |
| Rubble masonry . . . . . . . | $n = \cdot017$ |

The value given to $n$, the coefficient of rugosity, has a great effect on the discharge, so the selection of a suitable value in the case of river discharges is very essential. The coefficient in the case of natural channels will vary from $n = \cdot0250$ to $\cdot0350$.

(**39**) Methods of calculating flood discharges of rivers.

As already noted, the discharge of any channel is $A \times V$, in which $A$ is the area of the water section and $V$ the mean velocity of the current. The mean velocity can be deduced by actual observation of the surface velocity in feet per second, and the latter multiplied by a proper coefficient, which varies from ·65 to ·8 gives the mean velocity (*vide* par. 40).

In a wide stream several surface velocities can be observed, the water section being divided into any convenient number of parts and the floats are caused to pass through the centre of each of these divisions as near as possible; then the discharge will be the sum of these areas, each multiplied by the respective observed surface velocity, reduced to mean velocity.

A more accurate method is by means of vertical rod floats of various lengths. These can be either tin tubes weighted at the bottom so as to float nearly immersed, or else wooden rods with a piece of lead gas piping twisted round the bottom. The former are used in India, while the latter method, which is simpler, is recommended by Messrs. Tudsberry and Brightmore in "Water Works Engineering." These velocity rods should float just clear of the bed of the stream. If the latter is very uneven, as is often the case in rivers, the system will not work. The rods give the actual mean velocity at once and consequently furnish more accurate results than reduced surface observations.

Another method of obtaining actual mean velocity observations is by the use of current meters. The meter is immersed at $\frac{3}{5}$ the depth of the water in the middle of each division of the water area section, and the velocity is recorded automatically by an electric current, the meter being connected with a battery. A detailed description of Harlacher's Current Meter, with method of application, is given in "Water Works Engineering."

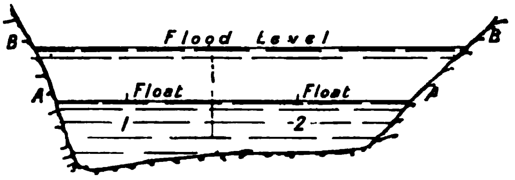


FIG. 6.

(**40**) To determine the mean velocity in a cross-section of a current where the maximum surface velocity is known, the following formula by Prof. von Wagner, gives reliable results:—

$$V = \cdot 7050 \times \cdot 003\, v^2 \qquad (17)$$

In which $V$ is the mean and $v$ the surface velocity.

Thus supposing the observed maximum surface velocity of a section to be 3 feet per second, the mean velocity $V$ will be $\cdot 705 \times 3 + \cdot 003 \times 9 = 2\cdot 115 + \cdot 027 = 2\cdot 142$ feet per second, *i.e.*, a little over ·7 $V$. Other authorities, as Jackson, give ·8 as the coefficient. In canal and river discharges in India ·7 or ·75 are the coefficients usually employed, so that the ratio given in Formula (17) can well be adopted.

(**41**) Fig. 6 is an imaginary cross-section of a channel. It is divided into two parts, separate surface velocity observations being supposed to have been taken in the centre of each division. The following form illustrates the method of arriving at the total discharge :—

| 1 Division. | 2 Observed Surf. Vel. | 3 Calculated Mean Vel. | 4 Area Measured. | 5 Discharge Col. 3 × Col. 4. |
|---|---|---|---|---|
| | Feet. | Feet. | Feet. | Feet. |
| 1 | 3 | 2·142 | 51·5 | 110·3 |
| 2 | 2·7 | 1·925 | 54·0 | 103·9 |
| | | | 105·5 | Total $Q = 214{\cdot}2$ cubic feet per second. |

(42) Supposing it is required to deduce the maximum flood discharge at a level higher than was the case when the velocity observations were taken, then the following formula will approximately determine the discharge required :—

$$Q = \frac{AqC\sqrt{R}}{ac\sqrt{r}} \text{ or } Q = \frac{Aq\sqrt{R}}{a\sqrt{r}} \times \frac{C}{c} \qquad (18)$$

in which $A$ and $a$ are the areas of the two water sections, $R$ and $r$ their respective hydraulic mean radii or $\frac{A}{WP}$, and $Q$ and $q$ the discharges in feet per second. $C$ and $c$ being the proper coefficients.

For example, in Fig. 6, supposing the flood discharge of the channel is required when the water is at the level $BB$, while the velocity observations have been taken when the water level was at $AA$. Then by reference to the Table in last paragraph, $q = 214{\cdot}2$, $a = 105{\cdot}5$, $wp = 32{\cdot}5$, and $r = \frac{a}{wp} = \frac{105{\cdot}5}{32{\cdot}5} = 3{\cdot}2$, and of the new section by measurement

$$A = 228 \text{ cubic feet, } WP = 43 \text{ whence } R = \frac{228}{43} = 5{\cdot}3 \text{ and}$$

$$\frac{Aq\sqrt{R}}{a\sqrt{r}} = \frac{228 \times 214{\cdot}2 \times 2{\cdot}3}{105{\cdot}5 \times 1{\cdot}79} = 595 \text{ cubic feet per second, nearly.}$$

Now the value of the coefficient $C$ is largely influenced by that of $R$, but slightly so by that of $S$, consequently if $S$ be known only approximately the fraction $\frac{C}{c}$ in which $S$ and also $n$ are common to both the numerator and the denominator can be obtained with sufficient exactitude without the precise value of $S$ being known. In the above let $n$ be ·0275, and let $S^{\circ}$ be assumed as ·2 per 1000, then from Table VII., Part 2, " Hydraulic Manual," p. 74, with $R = 5{\cdot}3$ $C = {\cdot}75$, and with $r = 3{\cdot}2$, $c = {\cdot}654$, the fraction $\frac{C}{c}$ will then be $\frac{75}{65}$, and $Q$ will be $595 \times \frac{15}{13} = 686$ cubic feet per second.

If the bed slope be assumed as ·3 per 1000 the fraction will become $\frac{725}{655} = \frac{145}{131}$ and $Q$ will become $595 \times \frac{145}{131} = 660$ cubic feet per second. This proves that for a very considerable difference in the value of $S$ or $S^{\circ}$, that of the resulting discharge is not so great proportionately.

In the example just given the whole water section has been dealt with, but greater accuracy would be obtained by having two values of $A$, $a$, etc., viz.:—those belonging to each of the two divisions in which the lower water section was divided.

(**43**) It often happens that actual velocity observations cannot be taken during a freshet, and there is no time available for putting up self-recording white gauge posts which will indicate the flood surface by the discoloration due to muddy water passing. In such cases the slope of the water surface being unknown, it can be assumed as parallel to the average bed slope, for which purpose a longitudinal section of the bed for at least a mile in length should be levelled and plotted, and at least three cross-sections taken of the channel showing the maximum flood level, which can be approximately obtained by observation of floating detritus deposited on the bank or on bushes, and by inquiry. These three cross-sections, taken at different places, should be plotted one over the other, and from this an average representative cross-section should be drawn. Having obtained $S$, *i.e.*, the sine of the bed slope, by levelling, $A$ and $WP$ are measured from the section and thus $R$ is obtained. The probable value of $n$, the coefficient of rugosity, has to be fixed with reference to the natural state of the channel, and by application of Formula (13), viz., $Q = 100\, Ac\, \sqrt{RS}$, the discharge is obtained. The value of the expression $100\, c\, \sqrt{RS}$ can be obtained from Jackson's "Hydraulic Manual" or other tables, in which Kutter's formula is worked out for English feet.

(**44**) For example, again utilising Fig. 6, supposing the value of $S$ obtained by levelling is $\frac{1}{3{,}000}$ or ·3 per 1,000, the known value of $r$ is 3·2, of $a$ is 105·5, whence the value of the expression $100\, \sqrt{RS}$ with $R = 3{\cdot}2$ and $S = {\cdot}30$ per 1,000 is found in Table VII., Part II., Jackson's "Hydraulic Manual" equal to 3·1 nearly. Again assuming $n$, the coefficient of rugosity, as ·0275; in Table XII., Part IV., the value of $c$ (with $R = 3{\cdot}2$ and $S = {\cdot}3$ per 1,000) is by interpolation ·654, whence the mean velocity is $(3{\cdot}1 \times {\cdot}654) = 2{\cdot}03$ feet per second, and the discharge $Q = AV = 2{\cdot}03 \times 105{\cdot}5 = 214{\cdot}1$ cubic feet per second, *i.e.*, identical with previous result. Such close coincidence would not be obtained in actual practice. When velocity observations are taken, the surface slope should usually also be obtained by carefully checked levelling, and noted in the discharge book. From the data thus available, viz., values of $S$, $R$, and $A$, the really correct value of $n$ which is applicable to this particular channel can be deduced, which will be most useful in future similar operations. The deductions can be made by trial and error, using the tables.

(**45**) The usual procedure of taking velocity discharges is as follows:—A convenient reach being selected, poles are erected in pairs at the starting and finishing points on either side of the channel, their direction being at right angles to that of the channel. The distance apart of these pairs of poles is generally 50 feet for a slow current and 100 feet to 150 feet for a more rapid stream. If more than one velocity observation is required in the cross-section, ropes should be stretched between the two pairs of poles clear of the water, on which the centres of the divisions, in which the water section is divided, should be clearly indicated by attaching a red streamer or otherwise. There should be two observers, one at each pair of poles, the third having the watch and record book. The floats, discs of wood for surface velocities, or the weighted rods as already described for mean velocity observation, are let go by an attendant at the proper place some short distance above the first pair of poles. When the float crosses the line of sight, the first observer calls out the number of the float, the discharge taker then enters the exact second and minute in the form book. The same is repeated when the floats cross the second line of sight. Stop watches are now invariably employed for this purpose, reading to half-seconds. At least three observations should be taken, the mean being adopted as the true velocity. If a float strays from the line it is expected to follow, that observation should be cancelled. Still weather is indispensable for surface observations. Two floats can generally be observed at short intervals, the second being dispatched before the first arrives.

**(46)** In the case just described the water section is first of all divided into divisions, but the most accurate method, particularly in taking large rivers, is to settle the width of the divisions in accordance with the actual course taken by the floats. The method of taking the discharges of the Nile illustrates this system, and the description obtained from "Egyptian Irrigation" is as follows:—

"A site was chosen where the river was fairly straight for fully 2 kilometres, the most uniform cross-section of the river was found by taking a large number of rough cross-sections. A peg was fixed in the bank opposite this section.

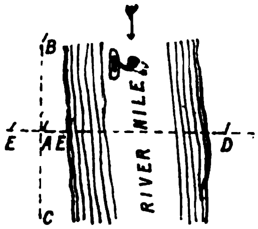


Fig. 7.

"In the accompanying plan, *A* is this fixed point.

"Through *A*, a line *B*, *A*, *C*, was marked, exactly parallel to the direction of the river, making *AB*, and *AC*, each equal to about the observed width of the river. A theodolite or plane table was put up at *A*, and the point *D* across the river on the line *AD* at right angles to *BC* was fixed. Since *AB* and *AC* were both capable of being measured, they were measured, and with the aid of the theodolite or plane table the length *AD* was obtained.

"In the line *AD* or its continuation, flags were put up at *E* and $E_1$ to direct the man in the steamer on the Nile. The theodolite or plane table was now put up at *B* or *D*, and observations made on a steamer or boat on the Nile, which as it got on the line *AD*, threw out a signal and took a sounding either with a line or sounding-rod. The boat or steamer traversed the whole section, and if any gaps were left without soundings, the plane table showed where they were, and they could be immediately filled in. While all this was going on, the nearest river gauge was being observed, and also a temporary gauge erected at the site of the discharge site. The two were afterwards connected by levelling. This completed the observations necessary for the cross-section, which was now plotted (*vide* Fig. 8).

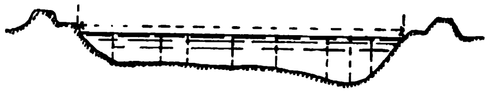


FIG. 8.

(**47**) "The surface velocity observations were now made.

"Two lines, *HJ* and *KL* (Fig. 9) were fixed, the former 50 metres up stream of *AD* and the latter 50 metres down stream. Between these lines the surface floats were observed. A theodolite or plane table was fixed at *H* and the boat or steamer was sent up stream with the floats. The observer stood at *H*, and another at *A*. The boat dropped a float into the stream at a distance of about 50 metres up stream of the line *HJ* at a convenient point, and the theodolite or plane table at *A* followed it until the man at *H* called out, on its crossing the line *HJ*. It was then observed and recorded. This was repeated as the float crossed the line *KL*.

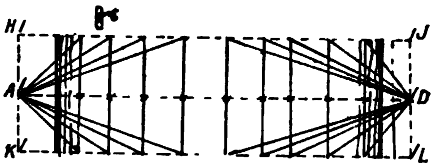


FIG. 9.

"The observers at *H* and *K* noted the time. When a sufficient number of well placed velocities over the half of the river near *A* were observed, observations for the other half were made from *D*. The field work was now over. On the cross-section, Fig. 10, the points *E*, *F*, *G*, and *C*, where the velocities were observed, were plotted.

"The cross-section was divided into a number of suitable sections, each ruled by one or more observed velocities. Each section was calculated separately, for instance, the section *XYZW*. Its area was *XYZW*, its wetted perimeter was *YZ*; its hydraulic mean depth was $\frac{XYZW}{YZ}$ and the rest followed from Bazin's coefficients and tables.

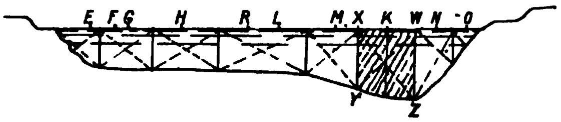


FIG. 10.

(**48**) "In addition to the discharges obtained from surface velocity observations, others were calculated from surface slope observations. A very careful cross-section of the river was taken as before, on a straight reach of some three or four kilometres, and at a point where the cross-section was fairly uniform, showing that the river here was flowing normally. A section taken in winter was found more accurate than in flood. There was enough water in the river to preserve its normal slope of water surface, and not enough to render the taking of soundings difficult. The water surface was compared with the nearest fixed gauge of the river. The section was very carefully plotted and the water surface drawn on it, with the corresponding gauge readings written against it. Horizontal lines (*vide* Fig. 11), one metre or half a metre apart, were then plotted on this section, corresponding to the different gauge readings. The cross-sections and hydraulic mean depths (*i.e.*, $A$ and $R$) were now calculated for each gauge reading. For the slope of the river surface a long reach of 50 kilometres was taken, as the chances of error were very much less than they would have been, if three or four kilometres had been taken. This was easily effected on the Nile, as there were carefully levelled gauges 50 kilometres apart.

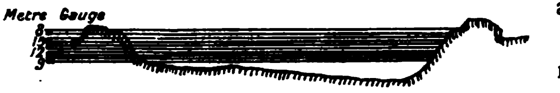


FIG. 11.

"The mean slope on this reach of 50 kilometres was taken as the slope for calculating the discharges. Of course on curves the slope varies, but since the cross-section had been taken in a carefully chosen normal site, the mean slope would refer to it. From the calculated velocities and discharges, a discharge and velocity diagram was made. This diagram was checked frequently by surface velocity observed as already described."

From the above the importance of taking the longitudinal section for some miles in length to ensure accuracy in the average surface slope, which is naturally variable, is demonstrated. The same would apply with equal force to a longitudinal section of the bed of a river, taken when dry if the water slope cannot be otherwise ascertained.

### *Afflux.*

(**49**) The calculation for afflux, *i.e.*, the rise of the water surface in a river due to an obstruction in such a weir is best arrived at by use of Tables II. and III. when once the value of the velocity of approach or $Q \div A_1$ has been decided on.

In a free overfall the afflux is the depth of film passing, plus that of the tail water surface below crest of weir. In a submerged fall, $H$ is the afflux. To obtain $d$, the flood discharge, which is a known quantity, should first be divided by the length of the weir, and the quotient will be the discharge per foot run. The next procedure is to find by interpolation in Table II. that value of $d$ which corresponds to the discharge and the velocity of approach.

(**50**) For example, supposing a weir to be built across the channel the section of which is shown in Fig. 6. Let its length be 30 feet and let the flood discharge of the river be estimated at 600 cubic feet per second. Now the area of the flood waterway is 228 square feet (par. 42), whence the mean velocity $V = \frac{Q}{A}$ and $Va$, the velocity of approach $= \frac{Q}{A_1}$. Assuming $A = A_1$ $Va = \frac{600}{228} = 2{\cdot}63$ feet. The unit weir discharge being 20 feet, by Table II., under $V = 2\frac{1}{2}$, we see that $d$ will lie between $d = 3$ and $d = 3\frac{1}{2}$, where the discharges in that column are 18·064 and 22·631 respectively. Their difference is 4·567, and that between the lower and the given discharge is 1·936 feet. Therefore the addendum to $d = 3$ will be $\frac{1{,}936}{4{,}567} \times \frac{1}{2} = {}·21$ nearly, and $d$ will $= 3·21$. Allowing roughly for the difference between 2·63, the actual velocity, and 2·5 which has been used, $d$ will probably equal 3 feet. The afflux will be 3 feet plus height of crest above tail water.

(**51**) If, however, the tail water is above the crest level of weir, and yet $H$ is greater than $\frac{d}{3}$, the fall is still considered as a free outfall, although actually submerged; under such circumstances the afflux equals $H$, *i.e.*, equals $d$ minus height of tail water above crest of weir, $d$ being obtained from Table II.

(**52**) The afflux in the case of a submerged weir where $H$ is less than $\frac{d}{3}$ is found by the same process, Table III. being used. Let us assume the crest of the weir to be 3 feet below the flood level of the river below the weir. The discharge is as before, 20 feet per foot run; $d$ will then equal $3 + H$. If $H$ be taken as 1, $d = 4$ and the corresponding discharge in Series 2, Table III., in column ($V = 2\frac{1}{2}$), is 28·89 cubic feet. $H$ must therefore be less than 1. If taken as ·5 the corresponding discharge of $d = 3\frac{1}{2}$ in Series I., Table III., is 16·72 cubic feet, which again is too small, hence $H$ lies between $\frac{1}{2}$ and 1, and as 20 is a mean between the two values 24·89 and 16·82 the value of $H$ may roughly be taken as ·75. This is probably quite near enough for most purposes. A more exact value can be found by utilising the intermediate Table for $H = \frac{3}{4}$. We have the initial values of the first four columns with $d = 3H = 2\frac{1}{4}$, the multiplicands for $V$ and the constants relating to columns 2 and 3, and with this information any discharge can be worked out with a very few figures. For example, in this case we require the discharge with $d = 3\frac{3}{4}$ and $V = 2\frac{1}{2}$. This value of $d$ is $1\frac{1}{2}$ units distant from the initial value, which is $3H$ or $2\frac{1}{4}$, and the expression in the second column will be $13·899 + (1\frac{1}{2} \times 6·9496) = 26·323$. The coefficient, in similar manner, will be $·8095 + (1\frac{1}{2} \times ·002632) = ·815$ and the product of columns 2 and 3 will be 21·400. This again multiplied by 1·06 the multiplicand for $V = 2\frac{1}{2} = 22·684$ cubic feet. This value is somewhat too high so that $H$ will be slightly below ·75, say ·7, which is as near the truth as possible. For practical requirements the afflux need only be calculated to the nearest half a foot. The above assume $Va$ equal to $V$. *Vide* para. 15, Chap. III.

### *Supply from run-off of Rainfall.*

(**53**) In the cases of storage reservoirs it frequently happens that the supply is obtained, not by one large stream which can be gauged and its minimum and maximum supply estimated, but by numerous rills and streams which cannot well be separately dealt with. Under such conditions the supply has to be estimated from the run-off of the rainfall, *i.e.*, the least and also the greatest supply available from the catchment area. This subject has been well treated in the "Madras Irrigation Manual," and as Southern India has a very extensive system of irrigation from storage as well as from direct canal irrigation, no better authority could be found. The following consists in great part of extracts from the above-mentioned work.

(**54**) *Rainfall.*—Rainfall is the source of all water used for irrigation purposes, and therefore a knowledge of its amount, character, seasons or periods, and of the effects produced by it, is of primary importance to all whose duty it is to design, carry out, improve or maintain irrigation works.

The resulting discharge from rainfall has to be considered in two ways: (1) the water to be utilised; (2) the water to be otherwise disposed of, and in connection with every irrigation work these two points require to be concurrently taken into account.

*Rainfall Registers.*—Rainfall registers are the foundation of a knowledge of the water resources of a country. It is desirable, therefore, that they should be kept in a convenient form at as many stations as possible; and, indeed, whenever there is someone available either permanently or temporarily to measure and record the rainfall. All old registers should be carefully preserved, abstracted and used as data. It should be remembered that an accurate record of the duration of all very heavy falls of rain is of much importance, and this should be impressed upon all in charge of rain-gauges.

(**55**) *Catchment Basin.*—The ground or country over which the rain falls and then drains off into the line of outfall or watercourse, is called the catchment basin; the boundary line of this basin is the watershed.

Every irrigation work is dependent for its supply of water upon the discharge, due to rainfall, from a basin of greater or less extent, varying in Madras from the 115,570 square miles of the Godavari basin above the anicut, to the fraction of a square mile supplying a small tank. The discharge of a catchment basin may be stored in tanks or reservoirs, in which case the proportion of the whole discharge intercepted may be large or small; or it may be utilised by means of irrigation canals or channels, with or without the aid of an anicut or weir, to raise the water-level in the watercourse, which may be the immediate source of supply. The water so drawn off may be used for what is termed direct irrigation, as in the supply of canals, or for that of tanks. In all cases, however, the whole quantity of water discharged by the basin has to be considered, and divided into two parts, the one to be used for irrigation and the other to be safely passed on or discharged to waste.

(**56**) *Maximum Discharge of Catchment Basin.*—Upon a right estimate of the greatest quantity of water liable to be discharged by a catchment basin will depend the safety of the works which may exist, or which it may be proposed to construct, to utilise a part of the water for irrigation purposes. An absolutely correct determination of this quantity is not possible, owing to the ever-varying circumstances under which heavy rainstorms occur; but nevertheless it is quite practicable, if a right use be made of available data, to form a fairly accurate judgment of the quantity of water which may have been disposed of, and then, by allowing a reasonable margin for errors in the estimate, the requisite safety may be secured.

**(57)** *Flood Data and Flood Discharge Formulas.*—The first data to be ascertained and made use of are the records of the levels and discharges of floods which have occurred in the past. These data require to be very carefully examined and checked by comparison of the evidence obtained at different points; and when it has been decided what, in the case of a particular discharge, the flood levels were, it is necessary to ascertain from the rainfall registers whether that discharge is likely to have approximated to the maximum to be expected.

Experience has shown conclusively that the flood discharge from large areas is proportionately much less than that from small basins. This fact has led to the introduction of formulas to assist in the determination of these discharges, and if judiciously used, they are of considerable value for this purpose and as a check on the results obtained from local observation and evidence.

**(58)** It should be borne in mind, however, that though such formulas may be confidently used for the practical settlement of questions connected with the design of works, when the conditions of their applicability have been determined on adequate data, they are otherwise to be relied on only as a guide to the reasonableness or the reverse of the conclusions independently arrived at. The formulas which have been in use are the following:—

(1) Ryves' formula . . $Q = k_1\,100\,K^{\frac{2}{3}}$ (19)

(2) Dickens' „ . . $Q = k\,100\,K^{\frac{3}{4}}$ (20)

in which $K$ represents the area of the catchment basin in square miles, $k$ is a coefficient depending for its value upon rainfall, soil, slope of ground forming the basin, etc., and $Q$ is the resulting discharge, which is usually taken in terms of cubic feet a second. The first of these formulas assumes that the discharge from catchment basins of differing areas varies as the cube root of the square of the area, while in the second the variation is supposed to be as the fourth root of the cube of the area. The general result is, in the former case a much more rapid diminution of proportionate discharge as the area increases than in the latter case. The data for determining which is the more generally applicable formula for Southern India do not as yet exist, and either may be usefully employed under the restrictions above noticed as necessary. The following additional caution should, however, not be lost sight of. No such formula can be strictly applicable with the same coefficient to areas of various sizes even in the

## TABLE VII.—($Q$) DISCHARGES IN THOUSANDS OF CUBIC FEET A SECOND.

| Areas in Square Miles. $K$. | $Q = k_1\ 100\ K^{\frac{2}{3}}$. | | | | | | | $Q = k\ 100\ K^{\frac{3}{4}}$. | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | COEFFICIENTS (RYVES') ($k_1$). | | | | | | | COEFFICIENTS (DICKENS') ($k$). | | | | | | |
| 1 | 0·200 | 0·300 | 0·400 | 0·500 | 0·600 | 0·700 | 0·800 | 0·200 | 0·300 | 0·400 | 0·500 | 0·600 | 0·700 | 0·800 |
| 5 | 0·584 | 0·876 | 1·168 | 1·400 | 1·752 | 2·044 | 2·336 | 0·668 | 1·002 | 1·336 | 1·670 | 2·004 | 2·338 | 2·672 |
| 10 | 0·928 | 1·392 | 1·856 | 2·320 | 2·784 | 3·248 | 3·712 | 1·124 | 1·686 | 2·248 | 2·810 | 3·372 | 3·934 | 4·496 |
| 25 | 1·710 | 2·565 | 3·420 | 4·275 | 5·130 | 5·985 | 6·840 | 2·236 | 3·354 | 4·472 | 5·590 | 6·708 | 7·826 | 8·944 |
| 50 | 2·714 | 4·071 | 5·428 | 6·785 | 8·142 | 9·499 | 10·86 | 3·760 | 5·640 | 7·520 | 9·400 | 11·28 | 13·16 | 15·04 |
| 100 | 4·308 | 6·462 | 8·616 | 10·77 | 12·92 | 15·08 | 17·23 | 6·324 | 9·486 | 12·65 | 15·81 | 18·97 | 22·14 | 25·30 |
| 250 | 7·938 | 11·91 | 15·88 | 19·84 | 23·81 | 27·79 | 31·76 | 12·57 | 18·86 | 25·15 | 31·43 | 37·72 | 44·01 | 50·30 |
| 500 | 12·60 | 18·90 | 25·20 | 31·50 | 37·80 | 44·10 | 50·40 | 21·15 | 31·72 | 42·29 | 52·86 | 63·44 | 74·01 | 84·58 |
| 1,000 | 20·00 | 30·00 | 40·00 | 50·00 | 60·00 | 70·00 | 80·00 | 35·57 | 53·35 | 71·13 | 88·91 | 106·7 | 124·5 | 142·3 |
| 2,500 | 36·84 | 55·26 | 73·68 | 92·10 | 110·5 | 128·9 | 147·4 | 70·71 | 106·1 | 141·4 | 176·8 | 212·2 | 247·5 | 282·8 |
| 5,000 | 58·48 | 87·72 | 117·0 | 146·2 | 175·4 | 204·7 | 234·0 | 118·9 | 178·4 | 237·8 | 297·3 | 356·8 | 416·2 | 475·6 |
| 10,000 | 92·83 | 139·2 | 185·7 | 232·1 | 278·4 | 324·9 | 371·4 | 200·0 | 300·0 | 400·0 | 500·0 | 600·0 | 700·0 | 800·0 |
| 25,000 | 171·0 | 256·5 | 342·0 | 427·5 | 513·0 | 598·5 | 684·0 | 397·6 | 596·5 | 795·3 | 994·1 | 1.193 | 1.392 | 1.591 |
| 50,000 | 271·4 | 407·1 | 542·9 | 678·5 | 814·2 | 950·0 | 1.086 | 668·7 | 1.003 | 1,337 | 1,672 | 2,006 | 2,340 | 2,674 |

## TABLE VIII.—($d$) *i.e.*, RUN-OFF IN INCHES DEEP FROM AREAS IN SQUARE MILES IN 24 HOURS.

$$d = \frac{9Q}{242K} \quad K = \text{areas in square miles.}$$

| Areas in Square Miles. $K$. | $Q = k_1\ 100\ K^{\frac{2}{3}}$. | | | | | | | $Q = k\ 100\ K^{\frac{3}{4}}$. | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | COEFFICIENTS (RYVES') ($k_1$). | | | | | | | COEFFICIENTS (DICKENS') ($k$). | | | | | | |
| 1 | 7·43 | 11·45 | 14·87 | 18·59 | 22·30 | 26·02 | 29·74 | 7·43 | 11·15 | 14·87 | 18·59 | 22·30 | 26·02 | 29·74 |
| 5 | 4·34 | 6·52 | 8·69 | 10·86 | 13·03 | 15·20 | 17·38 | 4·96 | 7·44 | 9·92 | 12·40 | 14·88 | 17·36 | 19·84 |
| 10 | 3·35 | 5·03 | 6·71 | 8·39 | 10·06 | 11·74 | 13·42 | 4·18 | 6·27 | 8·36 | 10·45 | 12.54 | 14·63 | 16·72 |
| 25 | 2·54 | 3·82 | 5·09 | 6·36 | 7·63 | 8·90 | 10·18 | 3·33 | 4·49 | 6·66 | 8·33 | 9·98 | 11·65 | 13·33 |
| 50 | 2·02 | 3·03 | 4·04 | 5·05 | 6·06 | 7·07 | 8·08 | 2·79 | 4·19 | 5·59 | 6·99 | 8·38 | 9·78 | 11·18 |
| 100 | 1·60 | 2·40 | 3·20 | 4·00 | 4·80 | 5·60 | 6·40 | 2·35 | 3·52 | 4·60 | 5·78 | 7·04 | 8·12 | 9·20 |
| 250 | 1·18 | 1·77 | 2·36 | 2·95 | 3·54 | 4·13 | 4·72 | 1·87 | 2·81 | 3·75 | 4·69 | 5·62 | 6·56 | 7·50 |
| 500 | 0·93 | 1·40 | 1·87 | 2·34 | 2·80 | 3·27 | 3·74 | 1·57 | 2·35 | 3·14 | 3·93 | 4·70 | 5·49 | 6·28 |
| 1,000 | 0·74 | 1·11 | 1·48 | 1·85 | 2·22 | 2·59 | 2·96 | 1·32 | 1·98 | 2·64 | 3·30 | 3·96 | 4·62 | 5·28 |
| 2,500 | 0·54 | 0·81 | 1·09 | 1·36 | 1·63 | 1·90 | 2·18 | 1·05 | 1·57 | 2·10 | 2·63 | 3·14 | 3·67 | 4·20 |
| 5,000 | 0·43 | 0·65 | 0·87 | 1·09 | 1·30 | 1·52 | 1·74 | 0·88 | 1·32 | 1·76 | 2·21 | 2·64 | 3·08 | 3·52 |
| 10,000 | 0·34 | 0·51 | 0·69 | 0·86 | 1·03 | 1·20 | 1·38 | 0·74 | 1·11 | 1·48 | 1·86 | 2·22 | 2·59 | 2·96 |
| 25,000 | 0·25 | 0·38 | 0·51 | 0·64 | 0·76 | 0·89 | 1·02 | 0·59 | 0·88 | 1·18 | 1·47 | 1·76 | 2·06 | 2·36 |
| 50,000 | 0·20 | 0·30 | 0·40 | 0·50 | 0·60 | 0·70 | 0·80 | 0·49 | 0·74 | 0·99 | 1·24 | 1·48 | 1·73 | 1·98 |

same part of the country and within the influence of the same intensity of rainfall, unless the other circumstances, such as slope of the ground, description of the soil, etc., be approximately similar.

(59) The values of $K^{\frac{2}{3}}$, $K^{\frac{3}{4}}$ for areas from 1 square mile to 50,000 square miles are given in Table VII. The chief difficulty will be found in the selection of a suitable coefficient. For the comparatively limited areas in the coast districts, where the country is flat, and the drainage takes a longer time to run off, $k = 4$ or $5$ have been found to be suitable coefficients in Ryves' formula, and 6·75 is a suitable coefficient for limited areas near the hills.

(60) The two Tables herewith given are (1) the discharges in thousands of cubic feet per second, calculated from the two formulas, Ryves' $Q = k_1 100\ K^{\frac{2}{3}}$ and Dickens' $Q = k 100\ K^{\frac{3}{4}}$. In both the formulas for an area of one square mile the discharge is equal to the coefficient employed, *i.e.*, the run-off is equal to the whole rainfall. The first square mile is the base of the formulas; as the area increases, the proportion of run-off becomes less, in one formula as $K^{\frac{2}{3}}$ and the other as $K^{\frac{3}{4}}$. To utilise the Tables properly, *i.e.*, to find the proper coefficient to be used, the method in Madras is to adopt as base an area of 5 square miles, in which area the maximum recorded rainfall is precipitated; this may be anywhere in a catchment basin. It is rightly assumed that heavy cyclonic storms only occur over a very limited area. In Table VIII. the run-off from the areas in square miles is converted from Table VII. into inches deep of run-off. But 5 square miles being taken as the base, this run-off will be equal to the rainfall. The following example is from the "Madras Manual":—

(61) "For example, suppose the greatest recorded rainfall within or near a catchment basin under investigation to have been 11 inches. The nearest run-off and rainfall to this in the line of 5 square miles (Table VIII.) is 10·86 inches under coefficient 500 for Ryves' formula, and about midway between 400 and 500 for Dickens' formula. Were no other data available, either formula with the coefficient indicated might be used to obtain the approximate discharge from the catchment. Supposing the catchment area to be 500 square miles, the resulting maximum discharge would be

| | | |
|---|---|---|
| by Ryves' formula . | 31,500 | cubic feet per second (Table VII.). |
| „ Dickens' „ . | 47,500 | „ „ „ |

a very material difference. Suppose, however, for a part of the basin to be dealt with, say for 250 square miles, there should be a recorded maximum discharge of 20,000 cubic feet per second, and that the rainfall at the time this occurred was the highest on record, viz., 11 inches. These data would indicate Ryves' formula to be applicable with the coefficient 500, whereas if the discharge had been about 28,000 cubic feet per second, Dickens' formula with coefficient 450 would be more likely to give a correct approximation of the discharge of the larger basin."

(**62**) The following Table shows the coefficients arrived at in the Cuddapah district in Madras, and may be useful as a guide. How the coefficients were arrived at is not stated, but they do not apparently agree with the method just described.

TABLE IX.

| Group. | | Av. Rainfall 9 years. | Max. Rainfall in 24 hours. | Coefficients $k_1$. |
|---|---|---|---|---|
| 1st group | A<br>B | 33·74<br>32·40 | 3·50<br>4·25 | 2 flat.<br>2·5 mixed.<br>3 hilly. |
| 2nd group | C<br>D<br>E<br>F<br>G | 29·45<br>31·82<br>31·91<br>28·37<br>24·04 | 5·40<br>5·50<br>5·70<br>6·00<br>6·20 | 2·5 flat.<br>3 mixed.<br>3·5 hilly. |
| 3rd group | H<br>J | 28·25<br>28·44 | 7·60<br>8·41 | 3 flat.<br>3·5 mixed.<br>4 hilly. |
| 4th group | K<br>L | 34·35<br>40·37 | 10·45<br>13·45 | 4 flat.<br>4·5 mixed.<br>5 hilly. |

(**63**) In a large number of tanks in Bombay Presidency, some of very great water spread, the proportion of run-off was taken at one-quarter the average annual rainfall, and the length of the waste weirs calculated accordingly. Colonel Mullins, in the "Madras Irrigation Manual," is of opinion that this proportion is excessive, and that it should more correctly vary somewhat with the slope of bed of stream as well as being proportionate to the area of the catchment basin.

(**64**) The shape of the catchment affects the coefficient, and attempts have been made to modify the coefficient so as to take the latter into account. There is a great deal of literature on this difficult subject, but nothing of a definite character. The formula is $Q = k100 \frac{B}{L} K^{\frac{3}{4}}$ (21) where $B$ and $L$ are breadth and length of the catchment (*vide* Jackson's "Hyd. Manual," Chap. I.).

The minimum discharge from a catchment area may be considered as 10 per cent. of the average rainfall.

When the capacity of a reservoir is well below its supply, it will fill several times during the rainy season, and its useful capacity can be taken to be 1¾, its estimated storage capacity.

**(65)** SUMMARY OF FORMULAS.

| No. | Formula. | Subject. |
|---|---|---|
| 1 | $V = c\sqrt{2gh}$ | Velocity of jet from orifice. |
| 2 | $Q = cl\frac{2}{3}\sqrt{2g}\left(h^{\frac{3}{2}} - h^{\frac{3}{2}}\right)$ | Discharge through orifice. |
| 2a | $Q = cl\ 5\cdot35\ (h_2\sqrt{h_2} - h_1\sqrt{h})$ | „ „ „ |
| 3 | $Q = cA\sqrt{2gH}$ | Approximate „ „ |
| 4 | $Q = cl\ 5\cdot35 \left\{ (H\sqrt{H} - h_1\sqrt{h^1}) + \frac{3}{2}\sqrt{H}(h_2 - H) \right\}$ | Discharge submerged orifice. |
| 5 | $c_1 = c\sqrt{1 + \frac{h}{H}}$ | Modification of coefficient for velocity of approach. |
| 6 | $Q = cA\sqrt{2g(H + h)}$ | Approximate discharge to include velocity of approach. |
| 7 | $Q = 5\cdot35\,cld\sqrt{d}$ | Discharge over weir free overfall. |
| 8 | $Q = 3\cdot333ld^{\frac{3}{2}}$ | Same with coefficient ·623. |
| 9 | $c_1 = c\left(1 + \frac{h}{d}\right)^{\frac{3}{2}} - \left(\frac{h}{d}\right)^{\frac{3}{2}}$ | Modification of coefficient to include velocity of approach free overfall |
| 10 | $Q = cl[\sqrt{2gH} + \frac{1}{3}(3d - H)]$ | Discharge submerged fall $V = O$. |
| 11 | $Q = cl\sqrt{2g}\left[(d + h)\sqrt{H} + h - \frac{1}{3}\left\{(H + h)^{\frac{3}{2}} + 2h^{\frac{3}{2}}\right\}\right]$ | „ „ including velocity of approach. |
| 12 | $\frac{1 - c^{\circ}}{97H}$ | Constant increment in coefficient Table III. |
| 13 | $Q = \cdot3ld^{\frac{3}{2}}$ | Discharge notch free overfall. |
| 14 | $Q = 3\cdot53d\sqrt{d} \times l_1$ | Discharge through bridge or open sluice free overfall. |
| 15 | $Q = AV = 100c\sqrt{RS}$ | Discharge of channels. |
| 16 | $C = \frac{\sqrt{R}}{100n}\left(\frac{m + 1\cdot811}{m + \sqrt{R}}\right)$ | Kutter's formula for coefficient. |
| 17 | $V = \cdot705 + \cdot003v^2$ | Reduction of surface to mean velocity. |
| 18 | $Q = \frac{Aq\sqrt{R}}{a\sqrt{r}} \times \frac{C}{c}$ | Deduction of discharge of channel from that known at a different surface level. |
| 19 | $Q - k_1 \cdot 100K^{\frac{2}{3}}$ | Ryves' formula discharge off catchment. |
| 20 | $Q = k \cdot 100K^{\frac{3}{4}}$ | Dickens' formula discharge off catchment. |
| 21 | $Q = k \cdot 100\frac{B}{L}K^{\frac{3}{4}}$ | Discharge off catchment (Jackson). |

**(66)** In this chapter D'Aquilar Jackson's "Hydraulic Manual" is constantly referred to, and necessarily so, as his Tables for the discharge of open channels worked out by Kutter's coefficients are indispensable, whenever any such calculations are undertaken. No irrigation engineer should be without this book, *vide* remarks of the same tenor in "Irrigation Works in India," with reference to the Canal and Culvert Tables by the same author.

# CHAPTER VI.

## CANAL HEAD WORKS.

### PART I. SUBMERGED DIVERSION WEIRS.

(1) WHERE a portion of the volume of a river is diverted for purposes of irrigation, the Canal Head Works usually comprise the following distinct items:—

First. The weir across the river.
Second. The weir Undersluices.
Third. The canal Head Regulator or Head Sluices.

Before dealing with the design of these several works some introductory remarks having reference to canal Head works as a whole will be necessary.

(2) The first point to be determined, in deciding on the position of a canal take off from a river, is the most suitable site for the purpose.

The following remarks, excerpted from Mr. Buckley's valuable work "Irrigation Works in India," will explain the points to which attention must be directed in forming an opinion on this matter:—

"The selection of the site of the head works is often a matter of considerable difficulty. One of the most essential points to be first considered is the nature of the silt carried by the river, whether it is fertilising or the reverse, what proportion of it it is desirable to carry down the canal, and whether the soil in which the canal is to be cut can stand the velocity which can carry the silt. When the slope of the canal is determined, an idea can be formed from the levels of the country at what point the water can be delivered on the surface of the ground, and the area under command of different sites for the head works can be ascertained. The approximate length and depth of cutting of the unprofitable portion of the canal which lies between the head works and the first point of irrigation can also be roughly worked out, and it is necessary to do this, as the cost of this portion may often be very great. The height of the weir above the bed of the river should next be determined. This will depend on the depth of water required in the canal and the level of the canal bed with reference to it. Then the effect produced by weirs of various heights on the flood level of the river should be worked out, the maximum flood discharge must be ascertained from gauge readings, from cross-sections of the channel and known surface slopes, and from these data the afflux, or height by which the flood level will be increased by the obstruction caused by the weir, can be calculated. This point is of great importance, as upon it depends the necessity for constructing embankments to control the river above the weir, so as to prevent inundation and the possibility of the river outflanking the weir. The most suitable position as a rule for a weir and head works of a canal is on a portion of the river where the channel is straight, the velocity uniform, and the sectional area of the stream fairly constant. A narrow gorge of a river appears to have the advantage of cheapness, but it may prove the most expensive owing to the greater velocity and depth of the current necessitating a heavier section than usual. A particularly wide reach of a river has, on the other hand, the disadvantage that the average velocity being decreased the deposit of silt and sand is encouraged, and the bed of the river above the weir is likely to become so raised that difficulty will be experienced in keeping a channel open to the take-off of the canal. Another point of importance is to select a site as near as possible to a stone quarry, so as to diminish the lead of building materials."

(3) A canal, as a rule should take off on the "cutting" side of river. The channel here is generally deep and open in times of low supply, besides which the bank on the cutting side often consists of a high bluff of clay soil, so that the canal Head as well as part of the Undersluices can be founded on solid material, which will prove a great saving in the cost of the foundations of these expensive works. In some cases canals take off at both sides of the weir, as in that of the Dauleshwiram head works of the Godaveri Canals; the best site would then be, where high grounds exist on both banks.

If such a site is not obtainable, the Undersluices belonging to the head work on the low side should be designed of more powerful draft than those on the natural cutting side of the river, and further supplemented by training works above the weir so as to form and retain an open channel leading past the canal Head.

(4) The usually best alignment of a weir in connection with a river channel is at right angles to a long straight reach, but there are occasions when a skew weir is desirable, an example of which is given in the block plan of the Rupar head works, Fig. 17, Chap. VII.

Where the river is of abnormal width, consisting of wide sand banks overflowed in flood time only, the channel can be narrowed with advantage, care being taken not to reduce the length of the weir below that of the average width of the river at sites where high friable banks occur.

This reduction in width will generally take place on one side only, viz., on the opposite bank to where the canal off-take is situated. The reduction in width is effected by the construction of a T head composed of earth or even sand of considerable dimensions and pitched with stone; an example of this is given in the Sidnai Canal plan of head works, in which the T head is shown (*vide* Fig. 16a). Up stream, training banks or spurs will also be necessary as a support.

(5) A difficulty experienced with weirs over very wide, sandy river beds, is the formation of islands of sand in the bed above the weir. These islands

get soon overgrown with weeds, and if not removed, form a serious menace to the safety of the weir by causing cross currents which, if they run parallel to the work, may undermine it from the rear. The best preventive of this danger is the adoption of collapsible iron shutters fixed on the crest of the weir; these are generally about 3 feet in depth and some 20 feet in length. The supports consist of iron rods hinged to the crest of the weir and also to the shutter at the height of the centre of pressure. Thus, when the river water tops the shutter crest, they turn over automatically and fall prone on the weir crest till raised again by hand. The leakage through them is insignificant. The weir then need not be built as high as would have been necessary prior to the introduction of shutters, and so does not offer near so much obstruction to the water section of the river when in flood. Any tendency to accumulation of sand in the bed above the weir crest level would also be swept away as freshets come down. In the Kistna Weir (a block plan of which is produced in Fig. 6d), which is not provided with shutters, extensive rubble spurs had to be run out and maintained upstream in order to guide the current straight on to the weir. These expensive training works would probably not be necessary at all, if collapsible shutters had been originally adopted in this old work.

In the Jamrao weir (Fig. 5) the collapsible crest shutters are 4 feet high and each 40 feet long. Some near the Undersluices are no less than 8 feet high, and in the Dhukwa Weir they are 8 feet high. These precedents prove that shutters deeper than the usual 3 feet can be successfully adopted, if circumstances warrant their use.

(6) Most river weirs are founded on sand alone and depend for their stability and staunchness against leakage on certain principles which will be shortly elucidated. In these investigations the quality of the sand, *i.e.*, its specific gravity, state of fineness or coarseness of its particles and the admixture of silt or mud with the mass, form considerations of great importance. The material of river beds may be roughly classified as follows:—(1) Fine light micaceous sands appertaining to the beds of the Indus, Ganges, Jumna, and other snow-fed Indian rivers. (2) The coarser, heavier sands in the rivers fed from sandstone ranges in Central India, such as the Godaveri, Kistna, Pennér and Colerún Rivers in the South of India, together with the Sôn and Mahanadi in the Province of Bengal. To this second category most river beds belong. The third class is a worse type than the first and is represented by the silt-laden sand of the Nile River. It is evident that the nature of the river bed must considerably affect the design of a weir, and this is proved by the section of existing works, which, although not originally designed on any fixed principles, as such were then unknown, still form precedents of safe construction.

(7) We will now proceed in the investigation of the frictional stability of a pervious substratum under pressure from a head of water, counterbalanced by the effective weight of the superimposed structure.

When the foundation of a work, which is subjected to the influence and pressure caused by a head of water, consists of a pervious material as sand, the status of the so-termed 'frictional stability' of the structure is a matter which demands attention.

This status of equilibrium can be defined as follows, viz., that the effective weight of the structure must be sufficient to compress the loose particles of sand which form the foundation of the work to such an extent, as to completely neutralise the effect produced by the pressure of water caused by the head, or difference of levels in the river above and below the obstructing weir.

This neutralisation is gradually effected by the hindrance to the creep or percolation induced by the superimposed weight acting on a sufficient length of base. The water pressure acts in a two-fold manner.

**(8)** First, longitudinally, by causing a current underneath the foundations of the work and thus gradually washing away its support; and, secondly, by the vertical upward hydrostatic pressure, which tends to lift the superincumbent mass and thus cause failure. These components of the water pressure are resisted, the first by the compression of the particles of the sand due to the weight superimposed, which should be sufficient to effectually neutralise the disintegrating action of the horizontal component, while the effective weight of the structure above directly resists the second upward vertical component.

In order to be able to design such works as river weirs, or any structure on a pervious foundation, on settled principles, it is clear that the value of the frictional stability of the various sands of which river beds are composed should be known. Up to a comparatively recent period no fixed principles existed whereby the efficiency or otherwise of different designs for river weirs could be tested, hence to assure success precedent had to be closely followed.

**(9)** The most primitive section of river weir adopted, of which there are numerous examples extant in India, is that of the anicut, or rapid, formed of loose rubble. These, which are mostly found in the Madras Presidency, consist mainly in a masonry breast wall founded on wells or blocks, sunk 7 feet into the sand below the average river bed, to which is added a rear slope of about one in four and a fore slope of one in twelve or one in fifteen, composed of loose rubble pitching intersected by two or more longitudinal walls of masonry, which are sometimes also founded on shallow wells. In addition, beyond where the toe of the fore slope meets the river bed, a talus of loose stone extends for a considerable distance and is generally of great depth. An example of such a mode of construction is given in Fig. 1 of the Sangam anicut over the Pennér River in the Madras Presidency (*vide* par. 64, *post*).

**(10)** Up to the year 1870, no weir of the anicut or rapid type had ever been constructed without one or more lines of undersunk square blocks or interlacing circular wells below the weir body, and although the exact value of these curtain wells from a hydrostatic point of view was not definitely known, still it was considered that they somehow formed an indispensable adjunct in the successful design of a weir. When the Okhla Weir over the Jumna River, the sand of which is of far inferior quality to that of the Madras and Bengal rivers, was designed by the late Colonel Rundall without any foundations below the normal bed of the river, its failure was confidently predicted by many experts. The work, however, proved a complete success and was found capable of upholding a head of 13 feet of water without sensible leakage or disturbance of the river bed below the work. The long cherished theory of the necessity of curtain protection was by this means completely demolished, and in addition, an object lesson was provided, from which it is possible to establish the value of the frictional resistance of a structure resting on a permeable stratum, as sand, on a definite basis of fact.

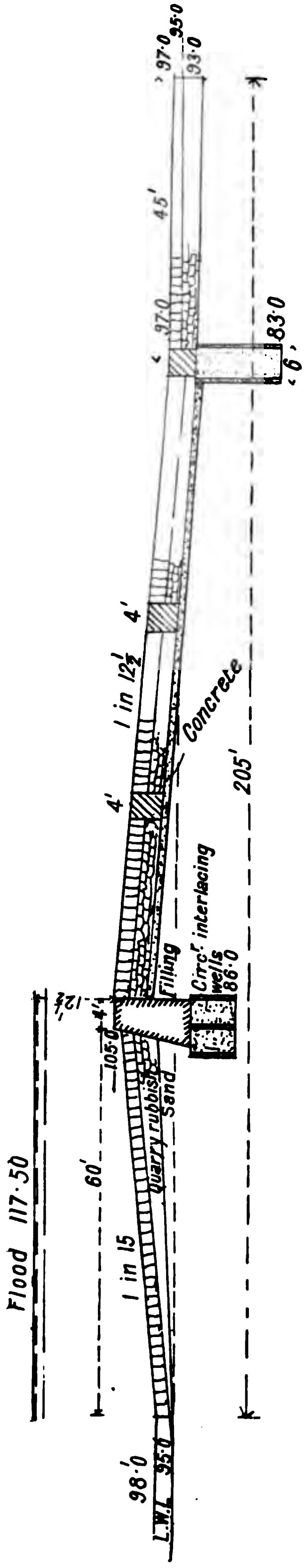


FIG. 1.—Sangam Anicut, Pennér River. Length, 4,076 feet; Discharge, 600,000 cubic feet.

(11) This was first put to a practical test by the Indian engineers employed in the Egyptian Irrigation Service in the design of the apron to the Grand Barrage over the Nile River.

The problem to be solved was the neutralisation of the effect of a head of water acting on the floor and foundations of the old barrage, without having to reconstruct this work, which was in an exceedingly shaky condition and quite incapable of service.

To effect this desired consummation, a long heavy apron of masonry was constructed up stream of the work and carried over the existing floor and some distance down stream. The principle on which it was designed was that the effective weight of the apron when submerged should be such as to completely neutralise the pressure due to the head of water imposed on the work.

(12) In this design, the Okhla Weir (Fig. 2) was adopted as the successful precedent to be followed. The rule governing the length and thickness, *i.e.*, the effective weight of the apron, was derived from the experience thus gained, and can be expressed in the following equation, viz.: $W = CHw$, in which $W$ is the effective weight per foot run, either in air or water, as

the case may be, of the mass of the structure, $C$ is a coefficient or multiplicand derived from the example of Okhla, $H$ the head of water acting on the work and $w$ the unit weight of water. $Hw$ is then the pressure of water per foot run.

If the mass is homogeneous, $W$ will equal $Aw\rho$ ($\rho$ being the specific gravity). Hence, eliminating the common factor $w$, the equation becomes simplified to $A\rho = CH$, whence $C = \frac{A\rho}{H}$. If the mass is not homogeneous, the formula will become $C = \frac{ZA\rho}{H}$, $Z$ being the sign of summation.

**(13)** It is evident that $C$ must vary with the value of the material of the river bed from a hydrostatical point of view. A value of $C$ having been obtained, suitable for the light sands of the rivers of the Ganges and Jumna Doab, it is an easy matter to deduce the altered value for other rivers, by increasing or reducing the coefficient in correspondence with the improvement or otherwise of the material of the bed.

**(14)** The section of the Okhla Weir is given in Fig. 2 (*post*). The maximum head occurs when the water is held up to *R. L.* 662·30, *i.e.*, to the crest of the weir shutters, the channel below being empty. This head, measured to the floor of the structure, is 13 feet. This floor is coincident with the normal bed level of the river. The actual low water level is doubtless somewhat lower, but if the increased head due to this lower level be assumed as the maximum head, the weight of the sand lying between the low water level and the base of the weir, which is unsubmerged, will have to be added as an asset to the effective weight of the structure, and as this sand weighs more than water, and the length of this strip in feet is greater than the coefficient, it is clear that the advantage in increasing the head down to actual low water level will lie with the weir, and that the hydrostatic effect of the greater will be less than that of the smaller head.

**(15)** The designers of the barrage apron adopted a value of 40 for the coefficient for Okhla Weir; how this value was arrived at is a matter more of conjecture than certainty. But we know this much, that the maximum head was taken at 10, not 13 feet, and the rubble filling was assumed to be all submerged. A very low value must also have been assumed for the submerged specific gravity of the rubble pitching.

The area of the loose stone pitching is estimated to be 1,350 square feet per foot run. Neglecting the masonry walls, the specific gravity adopted for the submerged pitching works out as below. As we have seen,

$$C = \frac{A\rho}{H} \therefore \rho = \frac{CH}{A} = \frac{40 \times 10}{1350} = \cdot 3 \text{ nearly.}$$

**(16)** Now with regard to the correct value of $\rho$ for the submerged pitching.

The stone itself has a specific gravity of 2·6, when submerged in water its reduced specific gravity will be $\rho - 1$ or 1·6. As, however, 50 per cent. of the mass are voids, the value of the specific gravity of the mass will be half this, or $\frac{1{\cdot}6}{2} = {\cdot}8$.

(**17**) Another and a specious method of reasoning, which is clearly erroneous, is as follows:—

Specific gravity of stone is 2·6. The voids in the mass being 50 per cent., the value of $\rho$ for the whole mass will be reduced to 1·3. When this mass is submerged in water its reduced specific gravity will be $\rho - 1$ or $1{\cdot}3 - 1 = {\cdot}3$.

The fallacy underlying this calculation is the assumption that the material is a solid mass, the whole being subject to flotation. This whole area includes the voids, which are clearly not thus affected. The voids are filled up with water instead of air, but as this involves a rise in the water level and consequent diminution of the head, the one counterbalances the other, and this fact does not affect the situation.

(**18**) The reduction in effective weight due to any penetration or rise of water in the mass of the pitching lying below the weir wall is more than discounted by the correspondingly involved reduction in the head; consequently this rise of water level above the foundation, which is also somewhat indeterminate in value, is in all cases neglected from consideration and the mass is assumed free from the effect of any partial submergence.

(**19**) In estimating the effective area of the weir body to balance the water pressure, it is considered that the whole sectional area should not be included. It is evidently in excess of actual statical requirements, the extreme extension of the work transversely being due principally to hydrodynamical considerations. Of the total area, that of the rear slope, which is submerged, will be omitted, as well as that portion of the talus lying beyond the incidence of the toe of the fore slope with the base. Likewise the heavier masonry in the walls will be neglected. When this curtailment is effected a wedged-shaped outline ten feet high at one end with a slope of one in twenty will remain. The area of this portion is $200 \times 5 = 1{,}000$ square feet. The proper relation of base length to head is $l = \frac{200H}{13} = 16H$.

The area being free, its specific gravity, as we have seen, is 1·3;

Whence $A\rho = 1{,}000 \times 1{\cdot}3 = 1{,}300$.

Now $C = \frac{A\rho}{H} = \frac{1{,}300}{13} = 100$. *Vide* also Appendix and par. 7.

(**20**) The author has constructed crib weirs in Burma (*vide* Fig. 16) with hardly any rear slope or talus, the latter being gradually thrown in as required. The pitched fore slope was one in fifteen, and was devoid of masonry walls, so that the actual sectional area was the triangle contained between the fore slope, the horizontal base and the vertical back, which latter was but a plank wall.

This area proved sufficient as regards frictional stability, but there was no excess of material, as in another weir, where the fore slope was shortened to one in ten, the pitching sank and springs occurred in the body, which all pointed to deficient fractional stability. The coefficient of this sand, which was of heavier material than that of Okhla, works out as 75, and $l$ as 20 $H$.

**(21)** Having adopted a value of $C = 100$ for the micaceous sands of the rivers similar to the Jumna, viz., the Ganges and Indus, that for the Madras and Bengal rivers of coarse sand (and in this category most rivers are included) can be obtained by deduction. The fore slopes given to weirs over these rivers are generally one in twelve and occasionally one in fifteen. The latter fore slope will, however, be considered as representative of the area enclosed; this will give some allowance for the curtain walls employed in these sections, and $C$ will consequently be $\frac{Ap}{H}$, or under similar conditions as at Okhla, will be $\frac{150 \times 5 \times 1\cdot3}{13} = 75$, and the relation $l : H :: 12 : 1$.

For Nile silt, which is a worse material than the Jumna sand, the engineers in Egypt increased their coefficient from 40 to 50, and this was the coefficient adopted in designing the Barrage apron and the floors of the Assiût Regulators.

A similar adjustment will increase the coefficient for the Nile from 100 to 125, and the relation of length of creep to head should be :: 20 : 1.

**(22)** With the low coefficients used in Egypt, applied to the calculation of floor areas, the weight of the floor alone was taken into consideration, and for such they appear to have answered. As, however, the weight of the overlying structure is the main factor in ensuring frictional stability, the submerged floor of a work is not the only part to be considered. To be strictly logical, the weight of the superstructure, which is mainly free from submergence, is clearly a factor in establishing the equilibrium sought for.

If the weight of the superstructure were included in the calculation of frictional stability, the low coefficients of 40 and 50 would not be workable at all. Whereas, as will be seen from the analyses which will follow, the effective weight of every part of a weir or regulator is included in the calculation for floor area, with use of the higher coefficients adopted by the author, which are founded on a basis of actuality, not impossible hypothesis.

**(23)** Before proceeding to the practical application of the coefficients in the design of works, it will be necessary to elucidate another point of importance, and that is the value of curtain walls and of sheet piling as assets to the frictional stability of a work.

Hitherto, as far as the author knows, no definite value has ever been assigned to these adjuncts. It is clear, however, that no analysis will be complete without this matter being definitely decided.

The action of a line of sheet piling or of curtain walls on a pervious stratum subjected to pressure from a head of water is clearly to lower the point of application of this pressure to the base of the obstructing plane.

In this case, the creep or line of action of the water pressure will be downwards to this base, and then it will rise upward on the opposite side. With two lines of sheet piling, as shown in Fig. 2a, the line of least resistance would be that of the dotted line connecting the bases of the two lines. The direction of the pressure will not, however, follow this line, as if it did, it is clear that the influence of the sheet piling would be greater the further apart the lines are, which hypothesis is contrary to reason.

The line of creep must therefore rise up in some direction towards the base of the superimposed structure, at what angle is indeterminate, but the inclination will be something below the angle of repose of the material.

Under these circumstances it will be fair to assume that the downward and upward inclination of the line of creep is at 45°. Such lines are shown dotted in all the sections subsequently given.

**(24)** The statical effect of this lowering of the incidence of the water pressure in the substratum of sand, is evidently that of adding the effective, *i.e.*, submerged weight of the sand embraced within the lines of creep as an asset to the weir. This area, with sheet piling, is the square of the depth. In Fig. 2a the depths of the lines of sheet piling are 10 feet and 20 feet respectively; the values of $A$ in each case will then be 100 and 400.

In Fig. 3b the influence line of the sheet piling is greater on one side than on the other, and $A$ will be the sum of half the squares of the several depths, or $\frac{18^2}{2}$ on the left and $\frac{10^2}{2}$ on the right. Total, $162 + 50 = 212$.

The specific gravity of sand varies considerably; it is assumed as 1·5 for rivers of Class I., 1·6 for Class II., and 1·3 for Class III.

The asset of the frictional stability of Fig. 2b, due to the sheet piling, will be—

$$A\ (\rho - 1) = 212 \times \cdot 5 = 106.$$

**(25)** Thus we see that the effective value of sheet piling or curtain walls is more or less in the inverse proportion of the river coefficients, which is evidently a sound axiom. The increase of length of creep will be $d\ \sqrt{2}$ or $1 \cdot 4 \times$ depth.

With a heavy material the effect will clearly be much greater than with a light quick-sand, when the useful effect of sheet piling will be but slight.

With regard to curtain wells of undersunk blocks, if at all deep, these are practically supported by skin friction, and it is considered that their effective weight as masonry cannot be properly considered as an asset. Their influence as regards frictional stability is simply that of a double line of sheet piling, and regarded as such, their effective value will be their area, plus the lateral influence, multiplied by the value of $\rho - 1$, for sand only.

Thus in Fig. 4 (*post*), the effective asset of the fore curtain wells or $A\rho$, which are 20 feet below base of the floor, will be $(20^2 + 20 \times 8) \cdot 5 = 280$, and will neutralise a head of $\frac{A\rho}{C}$ or $\frac{280}{100} = 2 \cdot 8$ feet.

**(26)** Having settled the values of the coefficients of frictional stability and also the influence of curtain wells and sheet piling in the same direction, we shall now proceed to apply the above data to the analysis of different designs of river weirs, both of those types which have actually been constructed as well as of certain modifications of the same which it is proposed to introduce.

Types of weir sections may be classified as follows. Type A is the ordinary rapid or anicut exemplified in Figs. 1 and 2.

In type B, a direct overfall is substituted for the fore slope, the fore apron being horizontal with its surface level with the normal river bed. An example of this type is given in the Narora Weir, Fig. 3.

Type C is a suggested compromise between A and B. In this type, the general characteristics of type B are followed, viz., an overfall weir wall with horizontal fore apron; but to avoid the expense of construction under water, which is necessarily the case with this type, the horizontal floor or apron is raised above normal bed level. The depth of the overfall weir is therefore reduced by the floor thickness, and beyond the masonry floor the pitched talus is built on a slope until the normal river bed level is attained, when its continuation is horizontal. Type C is illustrated in Fig. 2a, and others to follow.

The fourth type, D (Fig. 2b), is identical in form with type B, with the exception that the only masonry work is in the weir wall and its foundations, the horizontal apron being formed of a thick layer of clay covered with pitching like type A, only reversed, the surface being horizontal but the base inclined. This arrangement avoids expensive unwatering operations, as the section is first dredged out to the correct shape, and the material then deposited in the standing water. The clay can be deposited below the foundation lines shown on the sections or above them, as may be deemed desirable with due regard to its protection.

**(27)** Weirs of type A have this in their favour: they can be constructed mainly by unskilled labour and are fairly economical, provided a quarry exists in close proximity to the work. On the other hand, the cost of maintenance, especially during the first few years after construction, forms a heavy annual charge, the talus in these works absorbing an immense amount of stone which sinks out of view and is not shown in any of the sections given, which thus do not properly represent the amount of material actually used in their construction and maintenance.

**(28)** This disturbance of the talus is caused by the great velocity engendered in passing floods owing to the contraction of the waterway. The sloping apron tends to maintain the high velocity of the fall for a considerable way beyond the breast wall, and to throw the onus of resistance on the talus, which has consequently to extend much further than what is necessary in the other types mentioned, and by constant renewals becomes eventually of considerable depth.

On the other hand, in types B and D the area of the waterway beyond the overfall is not diminished, and the depth thus afforded is valuable for the purpose of neutralising the velocity engendered by the film of water passing over the crest of the weir wall.

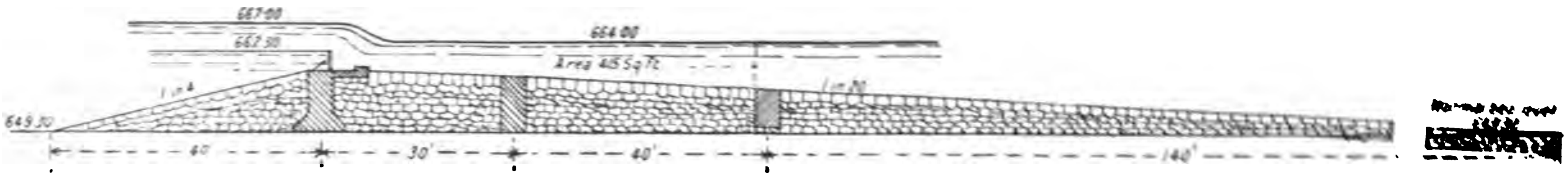
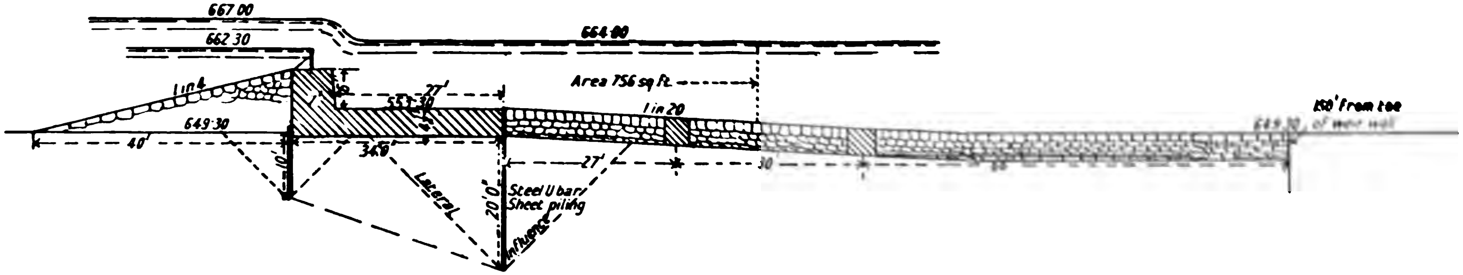
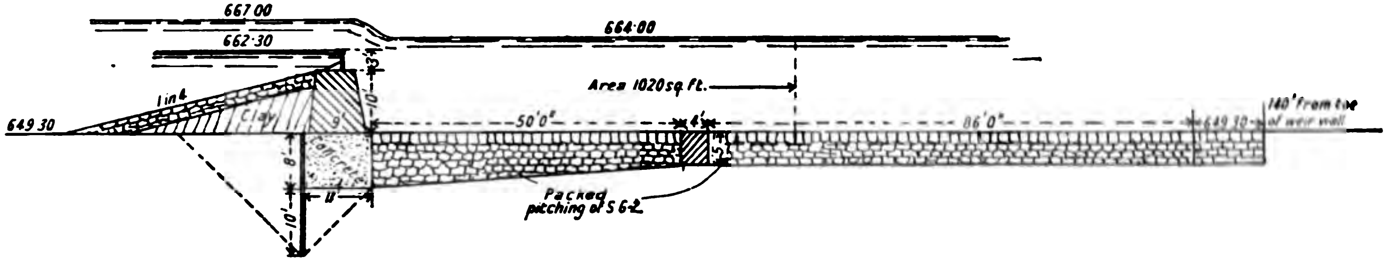


FIG. 2.



(2a, Type C.)



(2b, Type D.)

FIGS. 2, 2a, 2b.—Okhla Weir, Jumna River. Discharge, 150,000 cusecs. Sill: Agra Canal Head, 652·30, Sill Undersluice Head, 648·30.

**(29)** In Fig. 2 of the Okhla Weir (type A) the area of the waterway below the breast wall for a distance of 70 feet, is 435 square feet; in the alternative design of Fig. 2a of C type, the corresponding area is 756 square feet, while in Fig. 2b, the other alternative design in type D (also type B as regards outline), the area is 1,030 square feet.

The velocity of the current passing Okhla Weir (Fig. 2) has been ascertained by experiment to be as high as 18 feet per second at a point 20 feet beyond the breast wall. As velocities vary with the sectional areas of the waterway, the comparative velocities in the different sections will be as below:—

Type A Fig. 2 18 feet per second.
Type C Fig. 2a $18 \times \frac{435}{756} = 10{\cdot}3$ ,, ,, ,,
Types B and D Fig. 2b $18 \times \frac{435}{1030} = 7{\cdot}6$ ,, ,, ,,

This reduction in velocity must tell greatly in favour of types C and B, with reference to the amount of material in the talus, as also in the cost of maintenance.

**(30)** The objections to type B lie in the great expense inseparable from a masonry work built under water on a pervious foundation. This was accentuated in the case of Narora Weir (Fig. 3, *post*), by the lack of suitable material near the site, so that this work cost no less than two and a half times as much per foot run of weir as the Okhla anicut. However, under such circumstances no other type could well be used, except possibly type D, constructed, not of rubble over clay, but of grouted concrete slabs, which can be made at a distance from the site. Such a system of construction will be exemplified later. Fig. 3a is an alternative design for this work, the main difference being that steel sheet piling is substituted for the cumbrous and expensive well curtain protection peculiar to Indian works.

**(31)** The analysis of the section (Fig. 2a, Type C) will now be proceeded with, that of Fig. 2 being unnecessary, as it forms the basis on which the coefficient of 100 for this class of river bed was derived. Firstly, the head will be assumed as 13 feet, the masonry floor and most of the talus being thus taken as insubmerged.

The maximum head occurs when the water is held up by the shutters to *RL* 662·30, the river bed below being empty.

The pressure due to this head acts undiminished at the toe of the rear slope, to ensure which it must be rendered impervious by puddle.

The effective weight of the rear slope, which is submerged, reduces the effective head, so that at the inner side of the weir wall this reduction is equal to the area of the rear pitching, multiplied by its submerged specific gravity and divided by the coefficient 100, *i.e.*, $\frac{200 \times {\cdot}8}{100} = 1{\cdot}6$ feet.

The effective head at this point is thus reduced from 13 feet to 11·4 feet.

The weir wall is not submerged, and its specific gravity is $2\frac{1}{4}$, hence the further reduction due to it will be—

$$67 \times 2\tfrac{1}{4} = \frac{151}{100} = 1{\cdot}51.$$

Add to this the influence of the sheet piling, which is

$$\frac{10^2 \times {\cdot}5}{100} = {\cdot}5 \left\{(\rho - 1) \text{ for this sand being taken as } {\cdot}5\right\}$$

Total reduction 2·01 feet. The effective head at the toe of the weir wall will then be $11{\cdot}4 - 2{\cdot}0 = 9{\cdot}4$ feet.

This is a critical point in the design, as the thickness of the floor here must be sufficient to withstand the upward pressure of the effective head at this point.

The upward pressure, as we have seen, is 9·4 feet of water, the floor being $4\frac{1}{2}$ feet thick, with specific gravity of $2\frac{1}{4}$; the counteracting downward pressure is 10·1 feet of water, which is just sufficient. To continue, the reduction of head due to the area of the horizontal floor is—

$121{\cdot}5 \times 2\tfrac{1}{4} = 27{\cdot}3$ feet.

That of the sheet piling is $20^2 \times {\cdot}5 = 200$ „

Total 473 „

which divided by 100 is equal to 4·73 feet off the effective head, which will remain at this point at $9{\cdot}4 - 4{\cdot}73 = 4{\cdot}67$ feet. Now the pitched talus is free and partly submerged. The area of the free triangular portion is $\frac{70}{2} \times 4\frac{1}{2} = 157{\cdot}5$. This multiplied by $1{\cdot}3 = 204{\cdot}7$. To this will be added the submerged pitching for the same length, viz.: $157{\cdot}5 \times {\cdot}8 = 126$, making a total of 283·5. This divided by 100 reduces the effective head from 4·7 to 1·9 feet.

The additional length of pitching required for purposes of frictional stability beyond 70 feet below the fore curtain is obtained by the formula—

$$x = \frac{CH}{t \times \rho_1} = \frac{100 \times 1{\cdot}9}{4 \times {\cdot}8} = 60 \text{ feet } (t \text{ being thickness of floor, } \rho_1 = \rho - 1).$$

The length shown in the drawing is 150 feet from the toe of weir wall, *i.e.*, 63 feet beyond the previous point. As will be seen later, the length of the talus is not solely dependent on considerations of frictional stability, but must be a certain width for hydrodynamical reasons. In this particular case both closely agree. The value of $l$ is in excess of $16\,H$ (par. 19).

**(32)** In weirs of class C the effect on the floor as regards upward pressure is greater if the latter is considered submerged, though that on the whole work may be less. In such case the head in Fig. 2a will be reduced to 9 feet.

The analysis will then be as follows:—

| | $A$ | | $p$ | | $Ap$ | |
|---|---|---|---|---|---|---|
| Submerged rear slope | 200 | × | ·8 | = | 160 | Head, 9 feet. |
| Weir wall | 40 | × | $2\frac{1}{4}$ | = | 90 | |
| Submerged „ „ | 32 | × | $1\frac{1}{4}$ | = | 40 | |
| Sheet piling | 100 | × | ·5 | = | 50 | |

Total to toe of weir wall . 340 ÷ 100 = 3·4. Balance head, 5·6.

To meet this the downward pressure is $4\frac{1}{2} \times 1\frac{1}{4} = 5{\cdot}6$, *i.e.*, just barely sufficient.

To continue—

| | $A$ | | $p$ | | $Ap$ |
|---|---|---|---|---|---|
| Submerged floor . | 121 | × | $1\frac{1}{4}$ | = | 150 |
| Sheet piling | 400 | × | ·5 | = | 200 |

Total to end of floor . 350 ÷ 100 = 3·5. Balance, 2·1.

To meet this the downward pressure is $4\frac{1}{2} \times {\cdot}8 = 3{\cdot}6$ at end of talus.

The value of $ZAp$ for the pitching is $70 \times 4\frac{1}{2} + 63 \times 4 = 567 \times {\cdot}8 = 453$. This divided by 100 will neutralise a head of 4·5 feet, which is ·9 in excess of requirements. Thus the lower head has a greater effect on the floor but a less effect on the whole work. The value of $l$ exceeds requirements.

(**33**) The material used in the design 2a is evidently much less than that of Fig. 2, but the cost of the sheet piling necessarily used will tend to equalise matters. Interlocking channel bar sheet piling weighs about 36 lbs. per foot run, 1 foot deep; consequently the sheet piling in this design will weigh per foot run of weir 35 × 36 = 1,260 lbs., *i.e.*, over ½ a ton. This is estimated equivalent to the cost of 1,100 cubic feet of pitching material. The half bulb and cup pattern, however, weighs about 20 lbs. per square foot only, and is believed to be equally, if not more, suitable, as it can be stanched with cement grouting (*vide* Fig. 21).

At this particular spot, where an excellent stone quarry existed at the very weir site, the alteration of the section to one of type C might not possibly result in economy of first cost, but at other sites where such favourable conditions do not prevail, it would doubtless be found a much more economical design, particularly on a river whose coefficient is 75, where the fore sheet piling can be either dispensed with altogether, or used to a less depth.

(**34**) In Fig. 2b the section is of the D type. In this design half the rear slope is formed of clay with a high specific gravity, so that of the

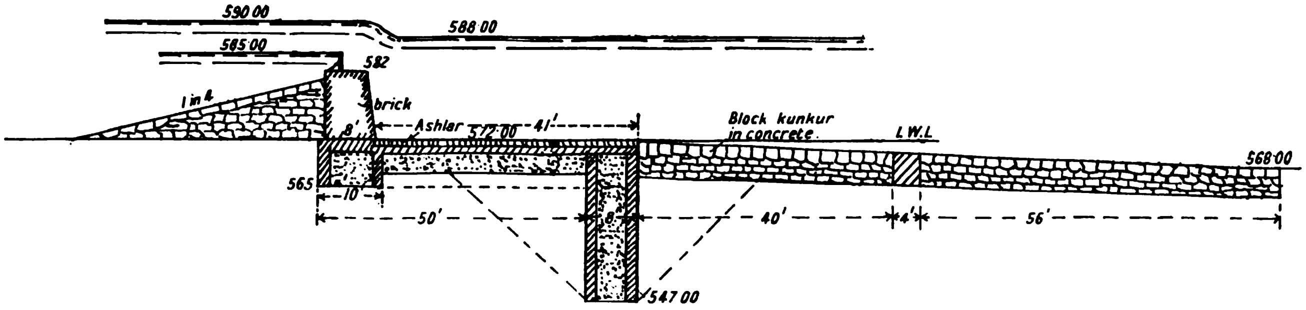
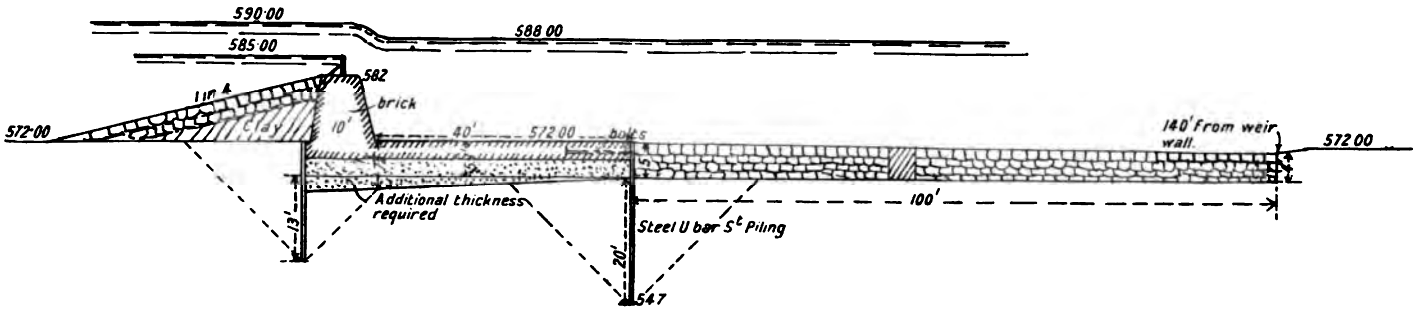


FIG. 3.—Type B.



(3a, Type D.)

FIGS. 3, 3a.—Narora Weir, Ganges River. Length, ¾ mile. Discharge, 300,000 cusecs. Sill: Canal Head, Rl.575·00, Undersluice, Rl.572·00.

whole, submerged, will be assumed as unity. We have, the head being 13 feet—

| | *A* | *p* | *Ap* |
|---|---|---|---|
| Submerged rear slope | 200 × | 1 | = 200 |
| Weir wall | 80 × | 2¼ | = 180 |
| Submerged „ „ | 88 × | 1 | = 88 |
| Piling | 212 × | ·5 | = 106 |
| Total to toe of weir wall | | | 574 |

This divided by $C = 5{\cdot}74$ ∴ Balance head $= 7{\cdot}36$ feet.

The downward pressure at this point is that due to 8 feet of material immersed, of specific gravity equal to unity, *i.e.*, 8 feet.

Continuing—

| | *A* | *p* | *Ap* |
|---|---|---|---|
| Fore bay (clay and stone) | 325 × | 1 | = 325 |
| Talus (stone) | 450 × | ·8 | = 360 |
| Total | | | 685 |

This divided by $100 = 6{\cdot}85$. Balance head, ·51 feet.

We see, therefore, that in this case the talus is somewhat too short. It should be—

$$\frac{100 \times 5}{5 \times {\cdot}8} = 12 \text{ feet longer.}$$

As a matter of fact, the specific gravity of submerged shallow pitching, faced on surface with heavy slabs closely set, will be at least 1, not ·8, which latter figure is suitable only for deep pitching; thus the length given in the drawing would be nearly sufficient.

The material in the apron is supposed to be composed of close stone pitching surfaced with slabs superimposed on a thick layer of clay.

(**35**) From all the foregoing it is evident that the rear slope and the weir wall itself are important items in a design, as it is their weight that reduces the head of water at the crucial point of the commencement of the floor in Fig. 2b. The rear sheet piling is also useful in this respect, besides being indispensable for constructional reasons, which is not the case in 2a. It is, however, not advisable to extend the rear slope beyond about 1 in 4, because the material placed here, though of great use, would be better employed in front of the weir, where it performs double functions, viz., dynamical as well as statical. A layer of clay will ensure its stanchness.

(**36**) We shall now analyse the section of the Narora Weir, Fig. 3, and the alternative design, Fig. 3a. This work, as already noticed, is of type B. Coefficient 100, $p = 2$, being brick, ashlar faced.

| | *A* | *p* | *Ap* | |
|---|---|---|---|---|
| Submerged rear slope | 200 × | ·8 | = 160 | Head, 13 feet. |
| Weir wall | 75 × | 2 | = 150 | |
| Submerged „ „ | 70 × | 1 | = 70 | |
| Total | | | 380÷100=3·8. | Balance, 9·2 feet. |

To counteract this we have a floor of 5 feet depth of brickwork and ashlar, with specific gravity of 1, leaving 4·2 feet upward pressure unbalanced. Even if the specific gravity of the material in the floor were taken at 1¼ submerged, there would still be a deficiency of effective weight. The thickness of the floor should have been at least 9 feet at this point, tapering up to 5 feet at its extremity.

To continue—

| | $A$ | $\rho$ | $A\rho$ |
|---|---|---|---|
| Apron . . . | 246 × | 1 = | 246 |
| Curtain . . . | 160 × | ·5 = | 80 |
| Lateral influence . | 400 × | ·5 = | 200 |
| Pitched talus, 5 feet deep . | 220 × | 1 = | 220 |
| ,, ,, ,, . | 280 × | ·8 = | 224 |
| Total . . . | | | 970 ÷ 100 = 9·70. Balance *nil.* |

Thus the total effective weight of the structure is just correct. After these calculations had been made, the author received information of a serious failure of this floor, which occurred some years ago. This fact is an excellent corroboration of the general correctness of the above analysis (*vide* Appendix).

**(37)** An alternative design for this work on the same lines is given in Fig. 3a. The same values for $\rho$ and $\rho - 1$ being also assumed, $\frac{A\rho}{100}$ up to the toe of the weir wall amounts in this case to 5·63 feet, leaving a balance of 13 − (5·6) = 7·4 feet. To counteract this upward pressure the apron floor is 7·6 feet thick. With $\rho - 1 = 1$, this is equivalent to 7·6 feet of downward pressure, which is just barely sufficient.

To the ordinary minimum depth of 5·1 feet, which is obtained by a formula to which allusion will subsequently be made, at the rear of the apron is added 2½ feet of increased depth, the base of the apron sloping upwards, as in sections of type D.

The rest of the calculations are as follows:—

The weight of the apron reduces the effective head of 7·4 to 4·8 feet at its termination, to meet which, neglecting the sheet piling, the resistance is 5·1 feet. Beyond, the talus extends for 100 feet, representing a reduction of head of 3·6 feet, leaving 1·2 feet unbalanced. Thus we see that the section as it stands is insufficient without the deep sheet piling, which has a neutralising value of $20^2 \times \cdot5$, or 2 feet, thus entirely disposing of the surplus. The value of $l$ is already sufficient without the aid of sheet piling.

**(38)** The expensive sheet piling could be dispensed with altogether if the masonry floor were continued a little further, or made thicker, but it is a most useful adjunct as a precautionary measure against failure of the talus.

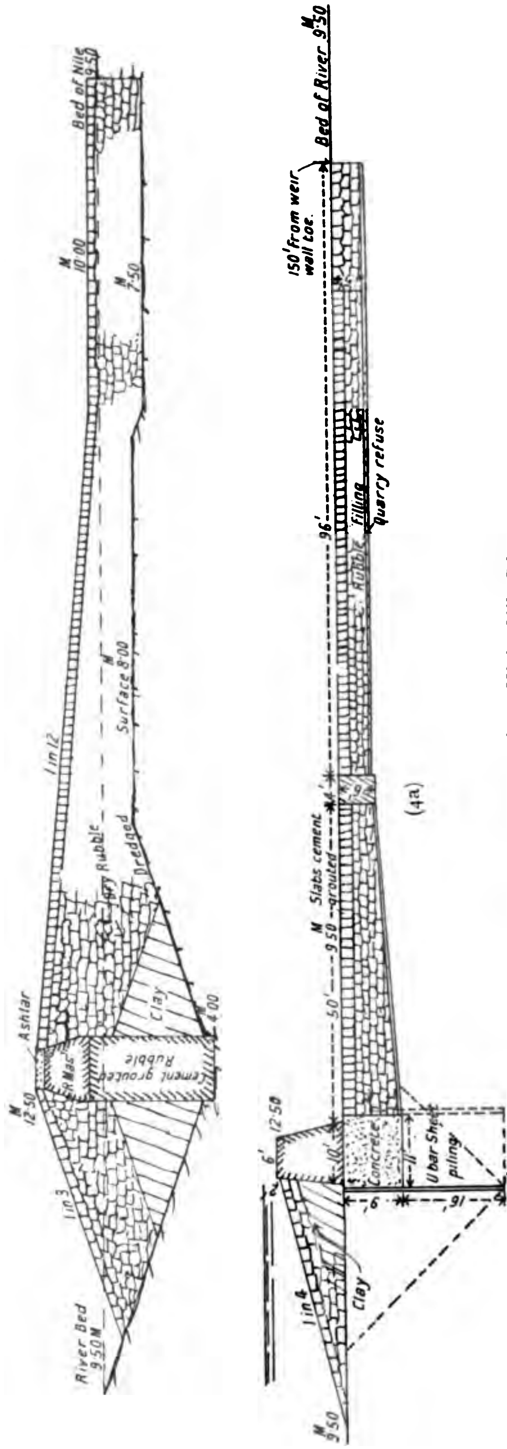
This alternative design would be much cheaper to construct than Fig. 3, as the curtains of steel, or concrete steel, sheet piling are far less expensive than those formed of blocks and wells, the sinking of which is a slow and costly process and greatly delays the completion of any work.

**(39)** In Fig. 3a the deep fore sheet piling is shown bolted back into the floor; this is to prevent it toppling forward if partially undermined owing to failure of the talus. Reinforced concrete sheet piling would perhaps be more suitable for the fore curtains in all weirs.

In the Narora Weir the construction of the heavy fore curtain and rear line, besides the cross lines of blocks, which are placed at intervals underneath the apron, must have been a very expensive and tedious operation. The adoption of steel or concrete sheet piling, the former now used extensively in the United States, would be much cheaper and would enable the work to be rapidly completed, thus saving interest on capital cost. The cross lines could be of temporary sheet piling dividing the area of the masonry apron into compartments of convenient size for unwatering operations. These could be withdrawn and used again as the work proceeded. The use of undersunk blocks or wells for curtain walls on shallow foundations was formerly necessitated by the general absence of pumping plant and of timber suitable for piling. The introduction of steel or reinforced concrete piling and the more extended use of modern facilities for unwatering operations will in time result in the old Indian system becoming quite obsolete, as has already been the case with the foundations of the recently constructed Assiût Regulators in Egypt.

**(40)** In this, as well as most weirs of type B, ashlar is largely used for coping to the weir wall and facing of the floor. The author considers this a useless extravagance. The sole advantage of ashlar lies in its high specific gravity, otherwise it is the worst possible material for the purpose, being very liable to be undermined, blown up and carried away. A homogeneous material, as hard rubble stone, set in hydraulic mortar and afterwards grouted with cement, would be much cheaper than ashlar and much safer. Where stone is unavailable near the site, as in the case of Narora, split slabs of hard stone set, some on edge and some on end, grouted with cement mortar, would have been an excellent substitute for the ashlar. They cost very little at the quarry, and there is not much waste, so that the carriage by rail to the site of the work would not be much more than that of heavy blocks of cut and squared stone, or else concrete slabs could be adopted.

**(41)** The section of the weir wall itself in Fig. 3 is too slight. As has been shown in para. 26, Chap. III., the condition usually prescribed, viz., that the incidence of the resultant line of maximum pressure must not fall outside the middle third of the base, has not been observed, probably with a view to economy. The wall, it is true, is safe from overturning, yet in this position weight is above all things required, and the weir wall section would have been better if built to the proper full size or even in excess of it. What is saved by skimping the section of the weir wall, which lies above water level, will have to be made good in the depth of the floor which lies below the water level, which is only half as effective from a statical point of view and is more difficult and expensive to build. This point is rectified in Fig. 3a.



FIGS. 4, 4a.—Damietta Weir, Nile River.

(**42**) The next work to be considered, viz., the Damietta Weir (Fig. 4), is a very remarkable one, the section being unlike any previously constructed. In spite of its very recent date, this mighty work cannot be commended as economically efficient.

The Damietta, as well as the similar Rosetta Weir, is built just below the Grand Barrage, of which mention has already been made, the object of these weirs being to further strengthen that decrepit structure. The crest is 3 metres, or say 10 feet high above Nile bed level, so that the bare head is 10 feet. Under these conditions, roughly half of the weir forward of the breast wall is submerged and half free. The value of $Ap$ is estimated to be 2,120, which, divided by the vertical head of 10 feet, would make the value of $C = \frac{2,120}{10} = 212$, whereas, as we have seen, the correct value of the coefficient for the Nile River is 125. The frictional stability of the weir is therefore very largely in excess of requirements.

(**43**) As will be seen by reference to Fig. 4, the profile of the weir is a combination of types A and D. The breast wall is of exceptional depth, being carried 18 feet below the river bed, and as it is composed of loose rubble grouted with cement by the Kinipple process, the cost of it must have been enormous. The probability is that this extreme depth was adopted in order to thoroughly protect the foundations of the barrage above, otherwise there can be no possible object in going to so great an expense. The main fault in the design of the profile is the unnecessary fore slope of the anicut type, which has been adopted.

**(44)** If this not only useless, but objectionable excrescence were removed altogether, the weir would cost much less and be really more stable, as the hydrodynamic force of the passing floods, the destructive velocity of which is aggravated by the congestion of waterway caused by the slope, would be greatly reduced were a direct overfall substituted. The outline would then be that of type D.

**(45)** This weir was the first constructed by the new method of dredging out the profile, without pumping the site dry, and of depositing material in the water standing in the hollow thus formed. The weir or core wall was enclosed in a temporary coffer dam, which was removed as the filling progressed. Clay was also deposited on either side, and subsequently the enclosed rubble filling was cement grouted in the water, under pressure. This method is doubtless very efficient and allows of rapid construction, which is always a great point, but it must have been very expensive. *Vide* "Min. Pro.Inst.C.E.," vol. clviii.

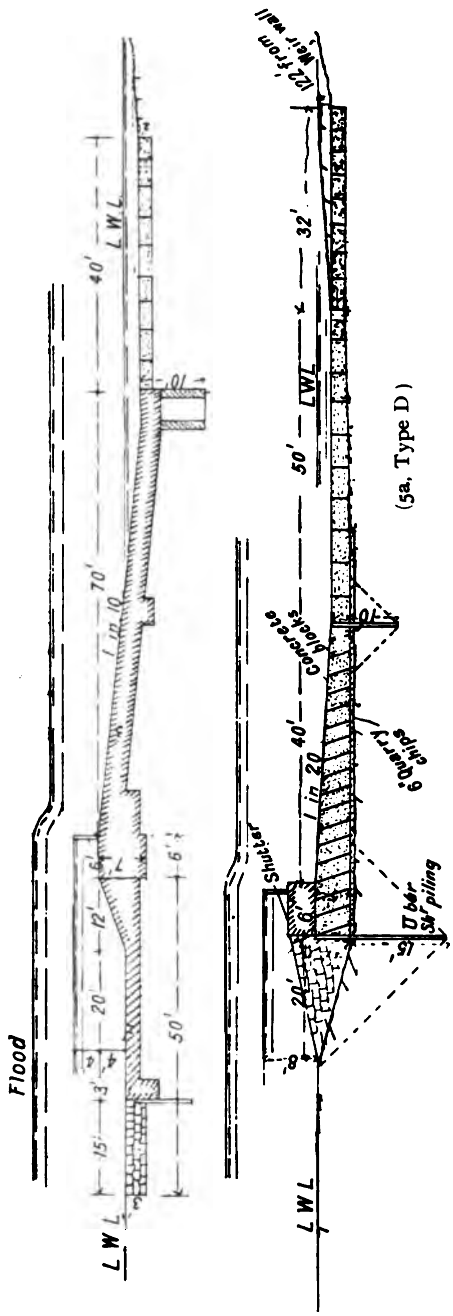
**(46)** Fig. 4a is a revised section for this work in type D. The usual row of deep sheet piling is adopted to the rear of the weir wall, as it is indispensable for constructional reasons. In case a deep solid curtain is really deemed necessary for the protection of the Barrage above, it could be formed by a double line of steel sheet piles, between which the sand and silt could be dredged out and concrete deposited in the water, when the piles could be withdrawn. This arrangement would save a large amount of dredging and stone and clay filling, which appears quite unnecessary. The second line of piles is shown dotted on Fig. 4a.

**(47)** The analysis of this design is as follows, the head being assumed at 12 feet, *i.e.*, 2 feet above the bare head, to allow for sudden freshets due to opening of the sluices above:—

| | $A$ | | $p$ | | $Ap$ |
|---|---|---|---|---|---|
| Submerged rear slope . . | 200 | × | 1 | = | 200 |
| Weir wall . . | 80 | × | 2¼ | = | 180 |
| Submerged „ „ . . | 100 | × | 1¼ | = | 125 |
| Sheet piling . | $\frac{625}{2}$ | × | ·3 | = | 94 |
| „ „ . | $\frac{256}{2}$ | × | ·3 | = | 38 |

Total to toe of weir wall . . . 637 ÷ 125 = 5·1 feet.

This leaves a balance of upward pressure of $12 - 5{\cdot}1 = 6{\cdot}9$ feet, which is met by 9 feet of pitching and clay with a specific gravity of ·9 or 8·1 feet.



FIGS. 5, 5a.—Jamrao Canal Weir (Indus inundation). (5a, Type D)

To continue—

| | $A$ | $p$ | $Ap$ |
|---|---|---|---|
| First panel of apron | 375 | × ·9 | = 338 |
| Second panel of apron | 500 | × ·9 | = 450 |
| Total | | | 788 |

$\div\ 125 = 6{\cdot}3$. Balance, ·6.

The pitching should therefore be longer by $\frac{125 \times {\cdot}6}{{\cdot}9 \times {\cdot}5} = 20$ feet nearly.

The specific gravity of the fore pitching with a proper admixture of small material, as gravel or quarry spalls, together with a surfacing of cement grouted slabs, is taken as one point above ·8, or as ·9.

In weirs of type D, in order to prevent leakage, which, though innocuous, may be considerable, the trapezoidal fore apron should preferably be composed of a thick lower layer of deposited clay with the pitching and surface slabs superimposed. To ensure protection of this clay layer it may be necessary to place it beneath the foundation line shown on the sections.

(**48**) Another recent example of weir construction which possesses some peculiarities worthy of notice is that of the Jamrao Weir built over the Naza, an inundation outlet of the Indus River. This section is illustrated in Fig. 5.

The weir is a low one, being only 4 feet high, but owing to the shutters, which are also 4 feet high, the head of water is 8 feet.

Under such circumstances, if the fore slope were of the usual pitching of low specific gravity, with its base level at normal bed level or low water level, the available depth of 4 feet would be insufficient to meet the upward pressure, if the ordinary four to one rear slope were adopted. Consequently the portion of the weir body to the rear of the breast wall has to be extended beyond what is usual, and further the apron itself has to be constructed of solid masonry of a higher specific gravity, in order to afford the requisite weight and stanchness. It is evident, however, that the material at the rear would be better utilised if it could be transposed to the front of the breast wall, as in the former position it is only useful from a statical point of view, whereas if placed in the apron or talus it performs a double function of resistance, viz., hydrodynamical as well as statical.

**(49)** In this case $C = 100$, $H = 8$, and $\rho = 2$, or submerged $= 1$.

With regard to submergence, the weir wall proper, *i.e.*, the 6 feet width, is in one piece with the rear slope, which is of impervious material, consequently the rear slope is an integral part of the weir wall and is not submerged above low water level. Whence all masonry lying above low water level will be free.

The value of $A\rho$, including the sheet piling 8 feet deep, is as follows:—

| | $A$ | | $\rho$ | | $A\rho$ | |
|---|---|---|---|---|---|---|
| Submerged rear pitching | 45 | × | ·8 | = | 36 | Head, 8 feet. |
| „ sheet piling . | 64 | × | ·5 | = | 32 | |
| „ rear floor . | 70 | × | 1 | = | 70 | |
| Rear slope . . | 20 | × | 2 | = | 40 | |
| Weir wall . . . | 24 | × | 2 | = | 48 | |
| „ „ submerged | 18 | × | 1 | = | 18 | |
| Apron to break . . | 21 | × | 2 | = | 42 | |
| „ submerged . | 14 | × | 1 | = | 14 | |

300 ÷ 100 = 3 feet. Balance, 5 feet.

The downward pressure at the break in the apron is 3 feet of brickwork with specific gravity 2, equivalent to 6 feet of water.

**(50)** If the whole work were considered submerged to crest level, the head will be reduced to 4 feet and the value of $A\rho$ will be 65 less, or 235; this, divided by 100, the quotient will be 2·35, leaving a balance at the break of the apron of 1·65 feet. To meet this the floor is 3 feet deep with a value of $\rho - 1$ of 1, equivalent to 3 feet of water downwards pressure, *i.e.*, more than sufficient. Thus we see that the section is less strained if fully submerged to what it is under the full head of 8 feet. As we have seen in Chap. III., the converse is the case when a weir wall subject to flotation alone is under consideration, and also when a weir is of type C.

The value of $A\rho$ for the rest of the work, with tail water at low water level, is 373 only, equivalent to a neutralising effect of 3·7 feet, leaving 1·3 feet unbalanced. An extension of the talus is therefore required.

The value of $A\rho$ up to the termination of the apron is 293, leaving a balance of 2·1 feet to be neutralised by the talus. If the blocks had been made 3 feet deep, the required length of the talus would be $\frac{100 \times 2{\cdot}1}{3 \times 1} = 70$ feet; it is actually 40 feet.

If the material of this weir had a higher specific gravity of 2¼, its sectional area as it stands would be sufficient.

(**51**) An alternative design for this work is shown in Fig. 5a. It is of type D, but with this difference, that the dredged base of the section is level, not inclined upwards down stream, while the surface is inclined downwards at 1 in 20. The body of the weir is composed of concrete slabs laid in the water at an inclination to the vertical, the slabs being slid one over the other, as was the case in the construction of the Colombo Breakwater. The interstices are supposed to be grouted subsequently with cement mortar under pressure. The drop wall is built of masonry on top of this trapezium of blocks. The rear slope is composed of clay covered by pitching, or else by gravel grouted pitching to raise its specific gravity to 2. The analysis is as follows:—

| | $A$ | | $p$ | | $Ap$ | |
|---|---|---|---|---|---|---|
| Rear pitching . . . | 90 | × | 1 | = | 90 | Head, 8 feet. |
| Sheet piling . . . | 230 | × | ·5 | = | 115 | |
| Weir wall . . . | 24 | × | 2 | = | 48 | |
| Submerged weir wall foundations . . | 30 | × | 1¼ | = | 37 | |

Total to toe of drop wall 290 ÷ 100 = 2·9. Balance, 5·1.

The downward weight is that due to 5½ feet of concrete with reduced specific gravity of 1¼, or 6·9 nearly.

To continue—

| | $A$ | | $p$ | | $Ap$ |
|---|---|---|---|---|---|
| Apron . . . . | 180 | × | 1¼ | = | 225 |
| Sheet piling . . | 64 | × | ·5 | = | 32 |
| Talus for 50 feet . | 138 | × | 1¼ | = | 172 |

Total . . . . 429 ÷ 100 = 4·3. Balance, ·8.

The continuation of the talus two feet deep will then be

$$\frac{100 \times \cdot 8}{2 \times 1\frac{1}{4}} = 32 \text{ feet, as shown in the figure.}$$

(**52**) The quantities of the two designs are as follows:—

| | Original. | Revised. |
|---|---|---|
| Pitching . . . | 45 | 90 |
| Sheet piling . . | 12 | 30 |
| Masonry . . . | 626 | 436 |
| Well sinking . . | 10 | *nil.* |

The masonry as actually shown on the plan in the original work is 496 feet; the difference of 130 square feet is what it would be if the conditions of frictional stability were satisfied as in the revised plan. If the specific gravity in the original masonry were raised to 2¼ from 2, the amount required would be about 500 square feet, *i.e.*, much the same as it actually is. As regards the cost of the two designs, that of type D will undoubtedly be much less, and, further, the stability of the talus of the new design with the direct overfall will be considerably greater, as the velocity of the current passing will be less, the waterway being less contracted. This hydrodynamical consideration is an important point as the talus in type A is greatly subject to displacement and failure and consequently has often to be renewed at considerable annual expense until equilibrium is eventually attained. In addition to this, in Fig. 5a the material is mainly disposed below the wall, and thus the section must possess greater resisting qualities to dynamical action than in the original work, where a large portion of the masonry, being situated to the rear of the fall, is useless for this purpose.

**(53)** The next example of the Kistna Weir (Fig. 6) is one also of type A. but contrasted with the last, is unusually high, the crest being 16 feet above low water level.

The usual height of river weirs built on sand is from 8 feet to 10 feet above normal bed level.

The profile of Fig. 6 is of the ordinary Madras type, with fore slope of one in twelve and the breast wall founded on a double line of interlacing circular wells, 7 feet deep below low water level.

The mass of material in this high weir is very great, and there is no doubt but that considerable economy would result in the conversion of the design into one of class C or D. The former is illustrated in Fig. 6b. This weir is not provided with shutters, so that the bare head to normal bed level is 13 feet. In every case it is deemed essential that the virtual head should be assumed as some 2 feet or 3 feet above the actual so as to allow for freshets. In this case 3 feet will be added, making a total head above low water level of 19 feet. As is invariably the case with type C, the action of the water pressure on the floor will be greater if the latter is assumed as submerged and the head reduced to 13 feet.

The value of $A\rho$ up to the toe of the weir wall is estimated to be 473. This divided by 75, the quotient will be 6·3 feet of upward pressure. This is met by an effective weight of $5\frac{1}{2}$ feet of floor, of specific gravity $1\frac{1}{4}$, equivalent to 6·87 feet of water; the difference in favour of the floor is thus ·57 feet of water. The value of $l$, inclusive of rear piling, is $12H$.

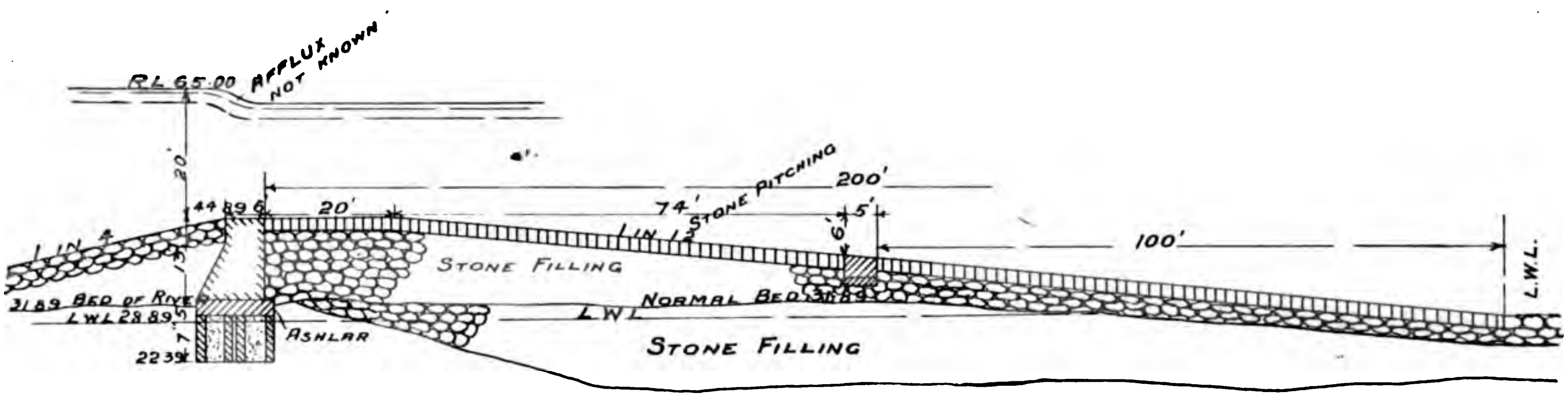
**(54)** If the head were taken as 16 feet down to normal bed level, the corresponding values would be—

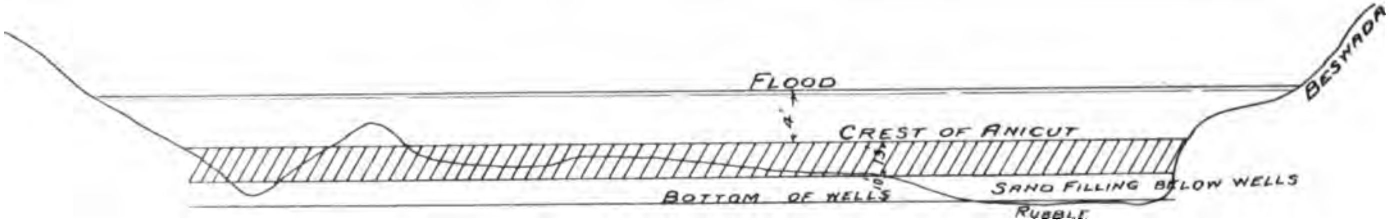
$$A\rho = 508 \div 75 = 6{\cdot}8. \text{ Balance, } 9{\cdot}2 \text{ feet.}$$

The counteracting force is 3 feet of masonry at $2\frac{1}{4}$, plus $2\frac{1}{2}$ feet at $1\frac{1}{4}$. Total, 9·37. Difference in favour of the floor, ·67.

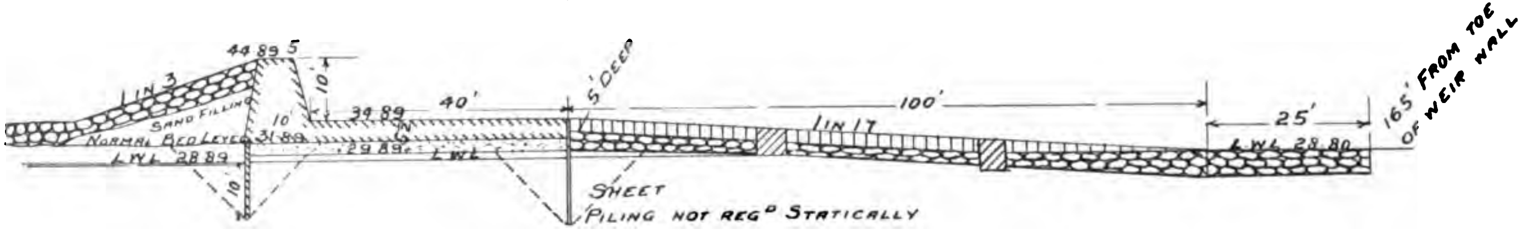
Thus, so far, the submergence of the floor causes a greater effect with the smaller head.

**(55)** The value of $A\rho$ of the floor, with head of 13 feet, is $220 \times 1\frac{1}{4} = 275 \div 75 = 3{\cdot}66$. Balance, $6{\cdot}7 - 3{\cdot}66 = 3{\cdot}04$. This is met by 5 feet of



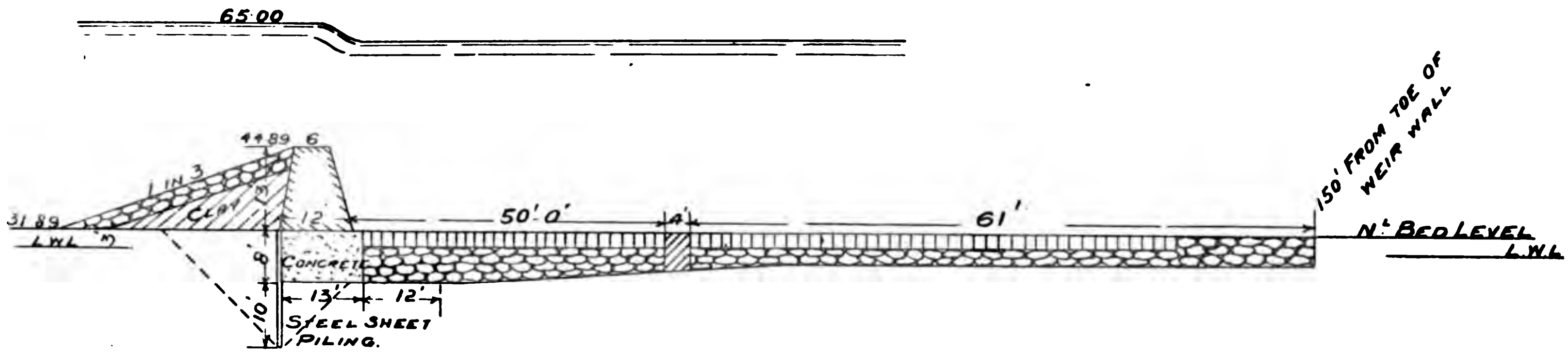


(6a, Section of river before construction.)

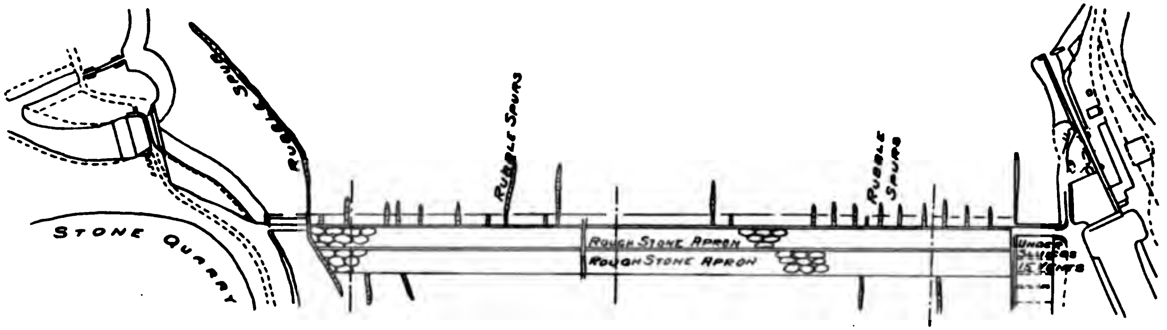


(6b, Type C.)

FIGS. 6, 6a, 6b.—Kistna Anicut. Discharge of River, 736,000 cubic feet.



(6c, Type D.)



(6d. Block plan of Kistna Head Works.)

FIGS. 6c, 6d.—Kistna Anicut. Discharge of River, 736,000 cubic feet.

pitching of specific gravity ·8, equivalent to 4 feet of water. Difference, 1 foot.

With the greater difference of levels of 15 feet, $A\rho$ to this point $= 395 \div 75 = 5·33$. The balance head will then be $9·2 - 5·33 = 3·67$. The counteracting force is 5 feet of pitching at specific gravity 1·3 or 6·5 feet. Difference in favour of floor, 2·83.

Thus we see all through the lesser head has the greater effect.

The talus for 100 feet has a mean depth of $\frac{5+4}{2} = 4\frac{1}{2}$ feet and the value of $A\rho$ is $450 \times ·8 = 360 \div 75 = 4·8$ feet, which more than neutralises the balance of 3·66.

The exact length of pitching required for statical considerations at a mean depth of $4\frac{1}{2}$ feet is $\frac{3·66 \times 75}{4·5 \times ·8} = 76$ feet.

**(56)** The lower curtain of sheet piling, usually necessary with rivers of coefficient 100 or 125, is not required for frictional stability, although it may well be inserted as shown in the drawing, as an additional precautionary measure. It will, however, add considerably to the cost.

**(57)** In Fig. 6c the section is in type D.

The floor level is placed at normal bed level; the head is thus 16 feet. Clay is used in the rear slope, raising the specific gravity of the submerged material to unity. The analysis is as follows:—

| | $A$ | $\rho$ | $A\rho$ | |
|---|---|---|---|---|
| Rear slope . . . | 260 | × 1 | = 260 | Head, 16 feet. |
| Weir wall . . . | 117 | × $2\frac{1}{4}$ | = 263 | |
| ,, foundation . | 104 | × $1\frac{1}{4}$ | = 130 | |
| Sheet piling . . . | $\frac{324}{2}$ | × ·6 | = 97 | |
| ,, ,, . . . . | $\frac{100}{2}$ | × ·6 | = 30 | |

Total to toe of weir wall . . $780 \div 75 = 10·4$. Balance, 5·6.

This is met by 8 feet of clay and pitching × ·8 = 6·4 feet.

To continue—

| | $A$ | $\rho$ | $A\rho$ | |
|---|---|---|---|---|
| Apron . . . . | 336 | × ·8 | = 280 | $\div 75 = 3·7$. Balance, 1·9. |

The length of pitching, 5 feet deep, to neutralise this balance head will be $\frac{75 \times 1·9}{5 \times ·8} = 36$ feet nearly. The value of $(\rho - 1)$ for the apron should be unity.

For hydrodynamical reasons this length is increased to 100 feet in the drawing. The depth of this talus would bear reduction to 4 feet.

With a high weir as this, the heavy weir wall and backing has a great effect in neutralising the head of water, for which reason a reduced rear slope of 1 in 3 has been adopted.

(**58**) The Kistna Weir was constructed as long ago as 1849, and was then considered a notable work. The fore slope of 1 in 12 then adopted has been religiously followed down to the present day, even in the very recent Damietta Weir, and still more recent, Laguna Weir in the United States (*vide* par. 29, Chap. VII.).

The longitudinal section given in Fig. 6a is useful as showing the amount of sand excavation and filling. In order to sink the rear curtain wells, the deep bed had to be filled up for the requisite width with sand, on which the wells were built in the dry, and through which they were subsequently sunk. Then the remaining width was filled up with loose stone. With the use of sheet piling the work could be carried out much more expeditiously. Where the low bed occurs a double row would be required, one near the end of the talus, and the intervening space could then be filled up to normal bed level with sand, thus saving much stone pitching. The channel below the weir would soon silt up of itself after the construction of the work. Sand forms an excellent foundation, if only protected from scour, and is often used to form part of the weir body in Madras works, notably in the Sangam and Adimapali Weirs (Figs. 1 and 10) (*vide* also Fig. 32c, Chap. XIII.).

The block plan in Fig. 6d shows the weir on plan with the rubble spurs up stream, placed in order to train the current straight on to the weir. The necessity of this would be largely obviated if collapsible crest shutters were used. The weir with its apron could then be reduced 3 feet in height, with great advantage in the cost of construction.

These shutters are not inserted in the alternative designs of Figs. 6b and 6c, the object being to exhibit variations in the design of the weir section under similar conditions.

(**59**) Figs. 7 and 7a are of the Jobra Weir over the Mahanadi River in Orissa, Bengal.

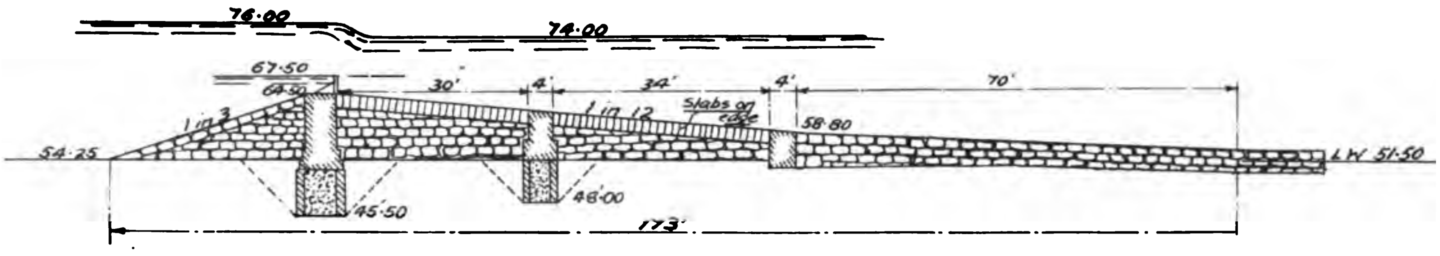
The section is of type A, of the common pattern, with the exception that two lines of curtain blocks are used. The analysis for frictional stability of the anicut, excluding the curtain, is as follows, rear slope taken as stanch :—

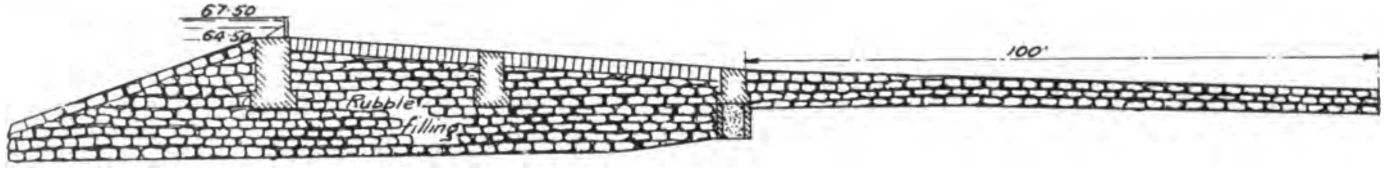
| | $A$ | | $\rho$ | | $A\rho$ | |
|---|---|---|---|---|---|---|
| Rear slope . . . | 200 | × | ·8 | = | 160 | Head, 13·15 feet. |
| Breast and longitudinal walls . . . . | 100 | × | 2¼ | = | 225 | |
| Pitching in talus and fore slope . . . . | 840 | × | 1·3 | = | 1,092 | |
| Total . . | | | | | 1,477 ÷ 75 = 19·7 feet. | |

Thus we see that the lines of curtain wells are not required for frictional stability, nor are they necessary for dynamical requirements.

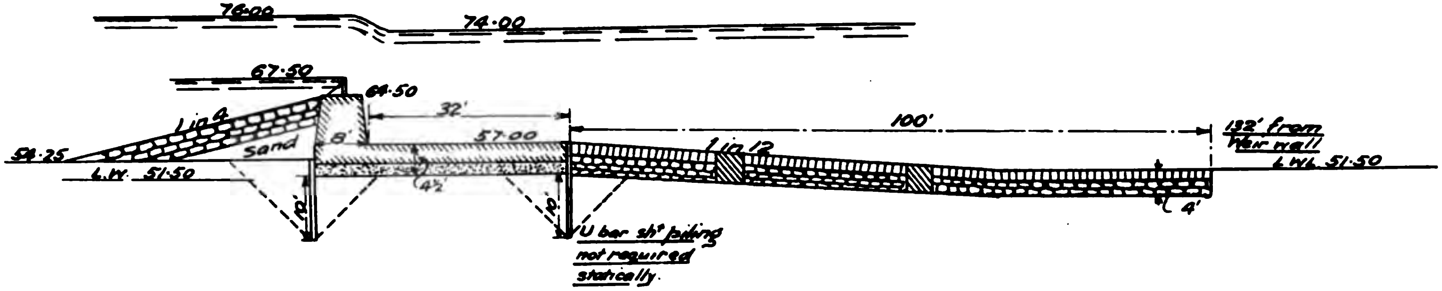
Fig. 7b is a design for the same work in type C.

The value of $A\rho$ up to toe of the weir wall is 380, which divided by 75 the quotient is 5·07. With a head of 10½ feet (tail water to floor level) the balance head here will be 5·43 feet.





(7a)



(7b, Type C.)

FIGS. 7, 7a, 7b.—Jobra Weir, Mahanadi River. Length, 6,400 feet. Discharge, 900,000 cusecs.

This is met by 4½ × 1¼ = 5·6 feet downward pressure.

The masonry floor further reduces this balance by 144 × 1¼ = 180 ÷ 75 = 2·4 feet, leaving 3 feet to be neutralised by the talus, the required length of which will be $\frac{75 \times 3}{4\frac{1}{2} \times \cdot 8} = 63$ feet.

In the drawing another 37 feet has been added for hydrodynamical considerations (*vide post*).

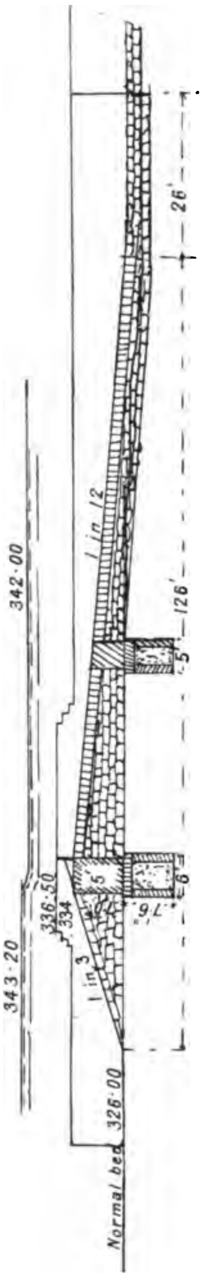


FIG. 8.—Dehri Weir, Sôn River. Length, 12,469 feet. Discharge, 830,000 cusecs.

**(60)** The section of the Dehri Weir over the Sôn River in Bengal (Fig. 8) is remarkable as being that of the longest weir in existence, its length being nearly 2½ miles.

The crest is 8 feet above normal bed level, and the shutters are 2½ feet high only. The statical head is therefore 10½ feet. The breast and longitudinal walls are founded on a continuous line of undersunk square blocks 7½ feet deep. The slope of the fore apron is 1 in 12, so that we see how careful the designers were not to deviate a hair's breadth from the old Madras practice.

The section is decidedly light owing to the low height of the breast wall, so it will be as well to work out its fractional stability without the curtain walls, as below, the stanchness of the rear apron being assumed:—

| | *A* | | *p* | | *Ap* | |
|---|---|---|---|---|---|---|
| Rear slope . . | 96 | × | ·8 | = | 77 | Head, 10½ feet. |
| Breast wall . . | 40 | × | 2¼ | = | 90 | |
| Apron . . . | 202 | × | 1·3 | = | 263 | |
| Longitudinal walls | 22 | × | 2¼ | = | 50 | |
| Apron . . . | 165 | × | 1·3 | = | 214 | |
| ,, submerged . | 105 | × | ·8 | = | 84 | |
| Total . . | | | | | 778 ÷ 75 = 10·4 feet. | |

The section, including the level talus beyond the termination of the slope, is therefore just sufficient for purposes of frictional stability.

**(61)** The expensive curtain walls nearly 5 miles long are therefore unnecessary. One row of rear sheet piling only would be desirable.

This weir is a contemporary of the Okhla work, built without any curtain protection whatever. Had Colonel Randall's design been followed in this case also, the saving effected would have been enormous.

The conversion of the profile into type D would undoubtedly result in economy of first cost and also subsequent maintenance.

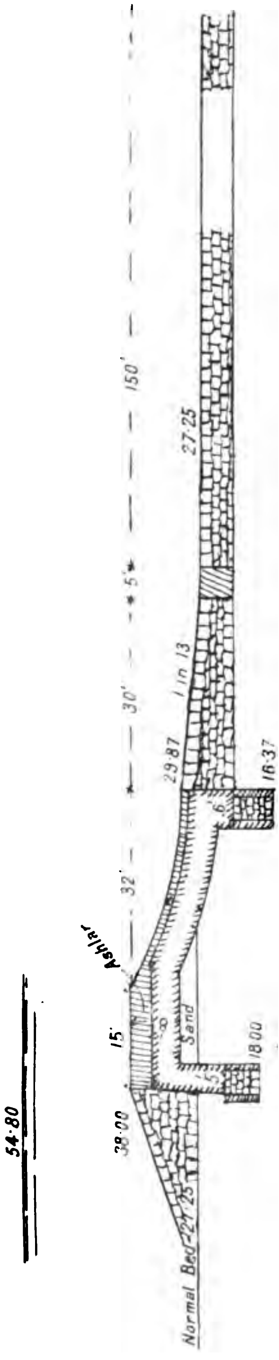
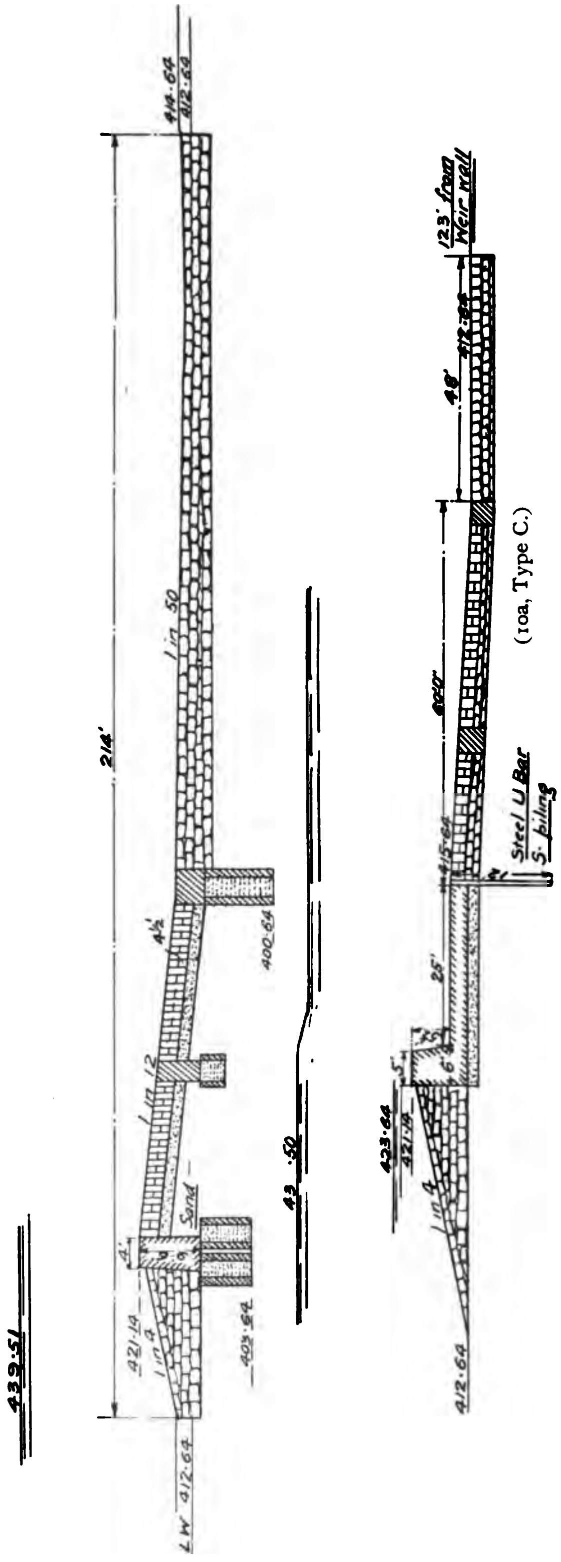


FIG. 9.—Dauleshwiram Anicut, Godaveri River. Length, 12,000 feet. Discharge, 1,210,000 cusecs.



(10a, Type C.)

FIGS. 10, 10a.—Adimapali Weir, Pennér River. Length, 2,243 feet. Discharge, 413,500 cusecs.

**(62)** A section of the Grand Anicut at Dauleshwiram over the Godaveri River is given in Fig. 9.

This work is remarkable as differing from the ordinary anicut type in many respects. It was constructed some time in the 'forties, when erroneous ideas regarding the destructive effect of water in a direct overfall were entertained, and it was deemed essential in the case of an overfall to resolve the current in a horizontal direction. The same error was perpetuated in the Ogee profiles of the falls in the old Bari Doab and Ganges canals, and is still rampant in the United States.

This weir consists of two breast walls founded on rows of circular 9 inch thick wells, sunk 6 feet in the sand and filled with rubble stone. These breast walls are 36 feet apart, and are connected by a wide horizontal crest and curved apron, bringing the level down to 8 feet below crest and to 2 feet above that of the normal river bed. Beyond this masonry apron is packed pitching, 30 feet in length, where a shallow masonry wall finishes the made apron. The talus of more or less hazard stone pitching extends for another 150 feet. The apron and horizontal crest are founded on sand filling, a system at once economical, and with this description of coarse sand, quite safe. This making up by sand filling is quite a common practice in Madras works, which might well be followed with advantage in many cases where rubble filling has been resorted to.

**(63)** It is evident that this section contains the disadvantages or weak points of both types A and B. The expensive masonry apron is quite equal in sectional area to that necessary in the floor of a direct overfall, while the contour of the profile must tend to increase the erosive action on the talus, which is a point it is desirable to avoid as much as possible.

The width of the weir is 230 feet, which is exceptionally great, being only exceeded by the 250 feet of the Okhla Weir, which work, however, is founded on sand of a very inferior description to that in the Godaveri River. The crest of this weir is 11 feet above normal bed level.

**(64)** The section of the Adimapali Anicut at the head of the Karnúl Canal, given in Fig. 10, is very similar to that of the Sangam (Fig. 1), both being constructed on the same river, the Pennér. From the R.L.'s on the plan it is evidently situated higher up than the latter weir, and so would be subject to heavy freshets. The weir body is partly made up of sand.

An improvement to this section, would be to convert the profile into type C, which has been done in Fig. 10a, and abolish the lines of interlacing curtain walls under the weir wall and in the centre of the apron, and substitute sheet piling for the rear curtain. The analysis of Fig. 10 is as follows—taking the head as 10 feet above L.W.L., *i.e.*, 2½ feet above weir crest.

| | $A$ | | $\rho$ | | $A\rho$ |
|---|---|---|---|---|---|
| Rear slope . . . . | 170 | × | ·8 | = | 136 |
| Breast wall . . . . | 39 | × | 2¼ | = | 88 |
| " " . . . . | 5 | × | 1¼ | = | 6 |
| Carried forward . . . | | | | | 230 |

| | *A* | | *p* | | *Ap* |
|---|---|---|---|---|---|
| Brought forward | | | | | 230 |
| Rear curtain | 80 | × | ·6 | = | 48 |
| „ lateral influence | 100 | × | ·6 | = | 60 |
| Sand filling | 62 | × | 1·6 | = | 99 |
| Apron | 125 | × | 2¼ | = | 282 |
| Longitudinal wall | 20 | × | 2½ | = | 50 |
| Curtain | 24 | × | ·6 | = | 14 |
| „ lateral influence | 25 | × | ·6 | = | 15 |
| Apron | 135 | × | 2¼ | = | 304 |
| Fore curtain | 55 | × | ·6 | = | 33 |
| Lateral influence | 121 | × | ·6 | = | 72 |
| Total | | | | | 1,207 ÷ 75 = 16. Balance *nil.* |

From this we see that the section is more than 1½ times in excess of what it need be up to the end of the apron. This weir, together with Fig. 1, virtually form a new type, the apron being composed of solid masonry built on made sand. It would clearly be an improvement to make the apron horizontal with a drop at the weir wall, as in type C.

(**65**) In the alternative design, Fig. 10a, the floor is made 4½ feet thick and 25 feet long, the overfall weir being 5½ feet high. The analysis is as follows, taking the head as 8 feet (floor submerged) and rear slope as stanch.

| | | | | | *Ap* |
|---|---|---|---|---|---|
| Rear slope | as before | | | | 136 |
| Weir wall | 28 | × | 2¼ | = | 63 |
| „ „ | 25 | × | 1¼ | = | 31 |
| Total | | | | | 230 ÷ 75 = 3. Balance, 5 feet. |

Floor is 4½ × 1¼ = 5·6 feet, downward pressure.

Continuation—

| | *A* | | *p* | | *Ap* |
|---|---|---|---|---|---|
| Floor | 112 | × | 1¼ | = | 140 |
| 80 feet of talus | 335 | × | ·8 | = | 268 |
| Total | | | | | 408 ÷ 75 = 5·4. Balance *nil.* |

Some sheet piling is, however, required, and a row 12 deep is provided at the end of the apron, having a value of $\frac{144 \times 6}{75} = 1{\cdot}1$ foot. The talus will require extension of another 48 feet to guard against erosion of the bed, as shown on the drawing. The piling would be better placed at the rear of the weir wall, and the floor somewhat lengthened. Here $l = 19H$.

*Comparative Statement of Quantities of Figs.* 10 *and* 10*a.*

| | Fig. 10 | Fig. 10a |
|---|---|---|
| Masonry | 100 | 193 |
| Pitched slope | 260 | *nil* |
| Rear and talus pitching | 676 | 392 |
| Curtain wells | 160 | *nil* |
| Steel sheet piling | | 16 feet. |

**(66)** The Brahmini Weir (Fig. 11) is an example of a low weir of a very economical section.

The value of $A\rho$, including the curtain walls, is $738 \div 75 = 9{\cdot}84$ feet of reduction of head.

The weir crest is 7½ feet above normal bed.

This requires increasing by an amount deemed a sufficient allowance for sudden freshets (there being no weir shutters); supposing 2 feet were allowed for these, the head would be $7\tfrac{1}{2} + 2 = 9\tfrac{1}{2}$.

The section is then exactly of the required dimensions, the curtains being necessary for frictional stability.

The talus doubtless extends beyond the incidence of the slope with the normal bed level, but this is generally omitted from the count of the effective area. If the slope of the apron were increased to one in twelve and the rear slope to one in four, the two masonry walls retained but the curtains abolished, the value of $A\rho$ will be 720, which divided by 75 will give 9½ feet head neutralised, exactly balancing the head.

This would cost much less than the existing section, and be just as effective. Until the Okhla Weir proved the contrary, these lines of curtain wells were deemed essential to successful construction. They greatly increase the cost of a work and are only twice as efficient as sheet piling of the same depth and in most cases are quite superfluous with river sands of coefficient 75.

**(67)** The section of the Barra Weir, like the Brahmini over same branch of the Mahanadi River, is given in Fig. 12. This is an actual example of type C, which was probably necessitated by the absence of cheap pitching material. Its analysis is as follows, head taken as 12 feet (rear slope stanch):—

| | $A$ | | $\rho$ | | $A\rho$ |
|---|---|---|---|---|---|
| Rear pitching . . . | 50 | × | ·8 | = | 40 |
| Weir wall . . . | 75 | × | 2 | = | 150 |
| Curtain . . . | 80 | × | ·6 | = | 48 |
| „ lateral influence | 36 | × | ·6 | = | 22 |
| Floor above wells . | 16 | × | 2 | = | 32 |

Total . . $292 \div 75 = 3{\cdot}9$. Balance, 8·1 feet.

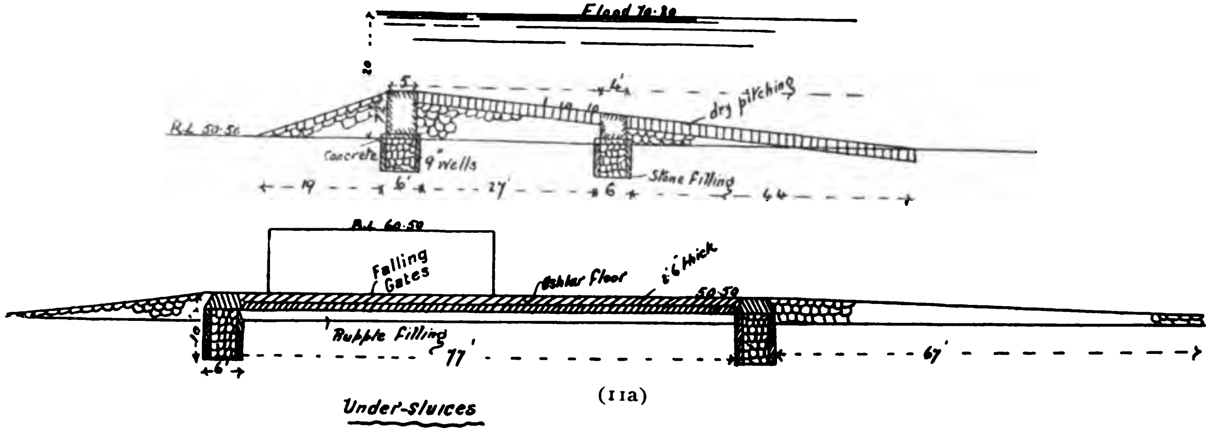
The downward counteracting weight is 4 feet × 2 feet = 8 feet.

If the floor is considered submerged the head will be reduced to 8 feet. The analysis of this same portion will then be:—

| | | $A\rho$ |
|---|---|---|
| Rear pitching . . . | — | 40 |
| Weir wall . . . | 45 × 2 = | 90 |
| „ „ . . . | 32 × 1 = | 32 |
| Curtain wells as before . | — | 70 |
| Floor . . . . | 16 × 1 = | 16 |

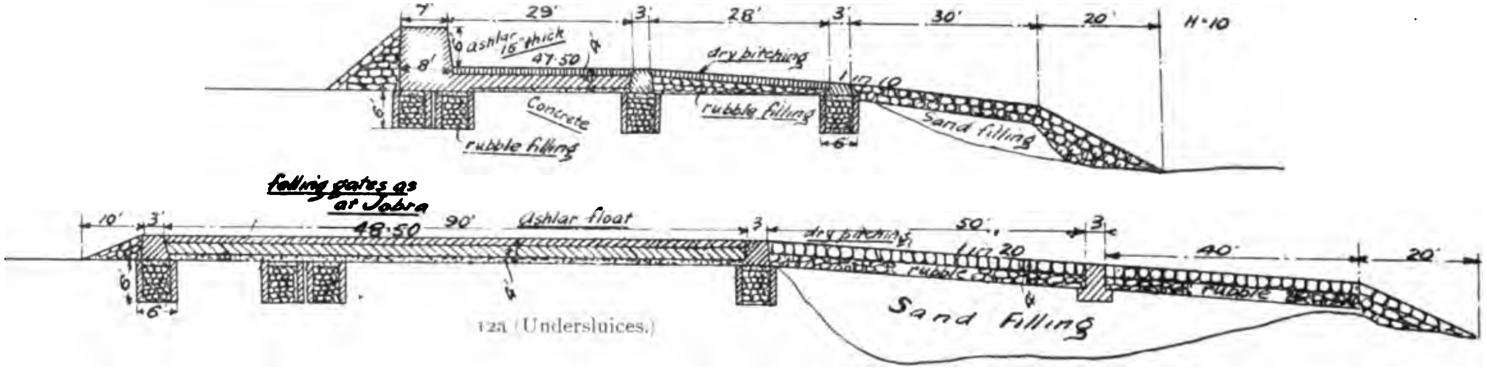
Total . . $248 \div 75 = 3{\cdot}3$. Balance, 4·7 feet.

The downward effective weight is $4 \times 1 = 4$ feet, leaving ·7 feet unbalanced.



(11a)

FIGS. 11, 11a.—Brahmini Weir.



12a (Undersluices.)

Figs. 12, 12a.—Barra Weir.

Thus we see that the effect on the floor, if this is considered submerged, is greater with a less head than if free with the greater head. This, as has been previously noted with regard to Fig. 2a, is invariably the case with type C, though as regards the whole section the effect of the greater head is frequently more.

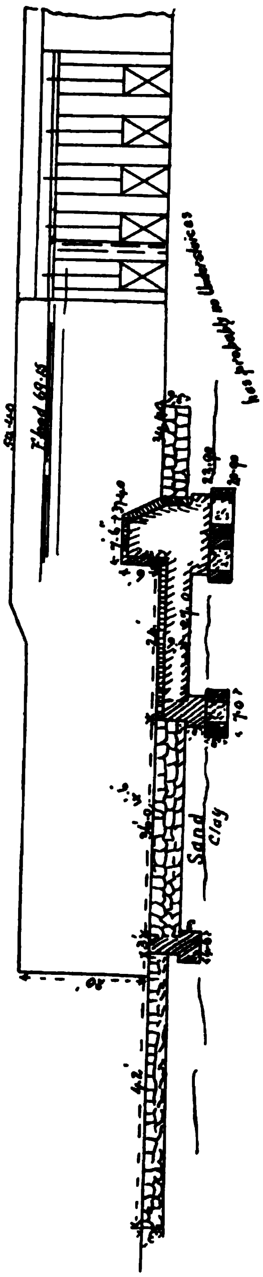


FIG. 13.—Srivakantham Anicut. Length, 1,380 feet. Discharge, 123,000 cusecs.

**(68)** From the above it is evident that the curtain wells in the design are not only necessary, but should have been deeper. The effective area required to be made up is $\cdot 7 \times 75 = 52\cdot 5$; with $\rho = \cdot 6$ the additional well area, including lateral influence, will be $\frac{52\cdot 5}{\cdot 6} = 87\cdot 5$ square feet.

The value of $A\rho$ for the existing curtain is 116, so that it has to be increased to $87 + 116 = 203$ square feet.

This will be arrived at by increasing the depth of the wells to 9 feet. Thus augmented, the curtain area $= 117 + 81 = 198$, which is near enough.

The rest of the section up to the third curtain has a value of $A\rho = 278$, which divided by 75 the quotient is 3·7 only. Deducting this from the 4·7 previously obtained, the deficiency amounts to 1 foot, equivalent to a value of $A\rho$ of 75. This is made up in the talus. The section would be better for the following alterations: Rear slope to be increased to 1 in 5; depth of floor and weir wall increased to 5½ feet at weir wall, tapering to 4 feet at end; a row of sheet piling 10 feet deep at termination of masonry apron; the pitching beyond to be 4 feet deep throughout; rear curtain and end curtain abolished. The rear slope to be made of clay puddle protected and weighted by stone pitching.

The talus of this work has evidently been washed away in a heavy flood, and the third curtain is the result of its renewal, as well as the sand filling in the hollow scoured out.

**(69)** The Srivakantham Weir (Fig. 13) has a much better section than the example already examined. The peculiarity of this work is the fact of its subsoil being sand only for a depth of 3 feet, below which is clay. The floor and pitching rest on the sand, while the weir wall and curtains are founded in the clay.

As the sand in the foundation is completely enclosed it forms an excellent bed for the apron, and the question of frictional stability does not arise, the actual base of the work being an impervious stratum, hydrodynamical forces having alone to be considered, and these can only be met by following successful precedent.

(70) The Pelandorai Weir (Fig. 14), over the Vellár River, is another specimen of overfall weir, which seems to be a kind of combination of types B and C. Originally designed as type B, an accident to the apron necessitated the drop given to the floor.

The weir wall is of extraordinary profile, bearing a close resemblance to an armchair, however, a heavy weir wall is necessary in this case, with the very thin floor provided. The actual overfall is 9 feet; if we add 3 feet, thus making the value for $H$ 12 feet, the floor thickness should be $\sqrt{H_1} + 1\frac{1}{2}$ or 5 feet, whereas it is only 3 feet. From the absence of the usual well foundations, the subsoil is probably clay, which accounts for the floor not having been blown up long ago.

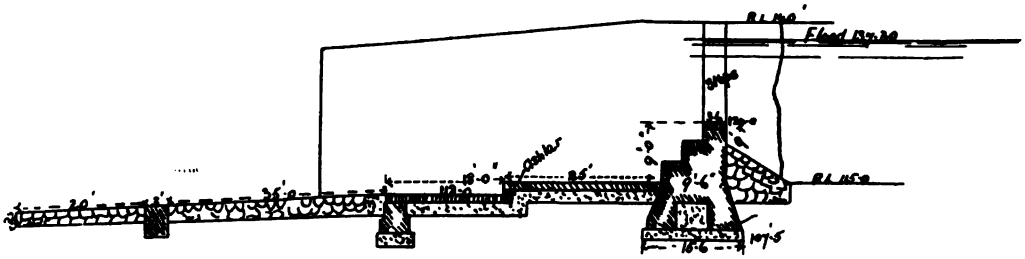


FIG. 14.—Pelandorai Anicut. Length, 860 feet. Discharge, 85,000 cusecs.

The value of $A\rho$ with upper apron just submerged and head equal to 12 feet, is as follows:—

| | $A$ | | $\rho$ | | $A\rho$ |
|---|---|---|---|---|---|
| Rear apron . . | 50 | × | ·8 | = | 40 |
| Weir wall . . | 54 | × | 2 | = | 108 |
| " " . . | 112 | × | 1 | = | 112 |

Total . . 260 ÷ 75 = 3·5 feet. Balance, 8·5 feet.

The counteracting weight is that of 3 feet of floor submerged, equivalent to 3 feet of water, leaving $5\frac{1}{2}$ feet unbalanced.

If the work were founded on sand the floor would require a thickness at the weir toe of $8\frac{1}{2}$ feet, or if only the bare height of weir be taken as limiting the head, $5\frac{1}{2}$ feet. The weir is evidently built on a clay foundation, otherwise the floor could not stand at its present dimensions.

(71) The Upper Colerûn Anicut (Fig. 15) was built as long ago as 1839. This weir is crossed by a road bridge of several spans about 25 feet in width. The bridge piers greatly obstruct the free passage of the current, as may be judged from the high afflux of $4\frac{1}{2}$ feet.

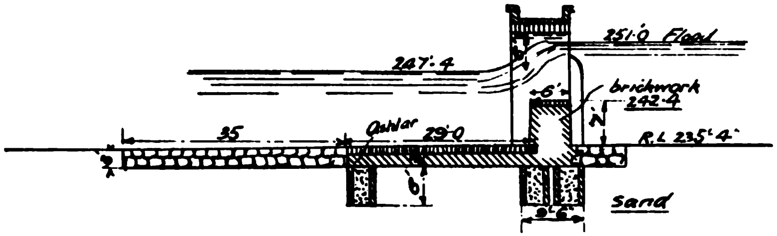


FIG. 15.—Upper Colerûn Anicut. Length, 2,929 feet. Discharge 284,000 cusecs.

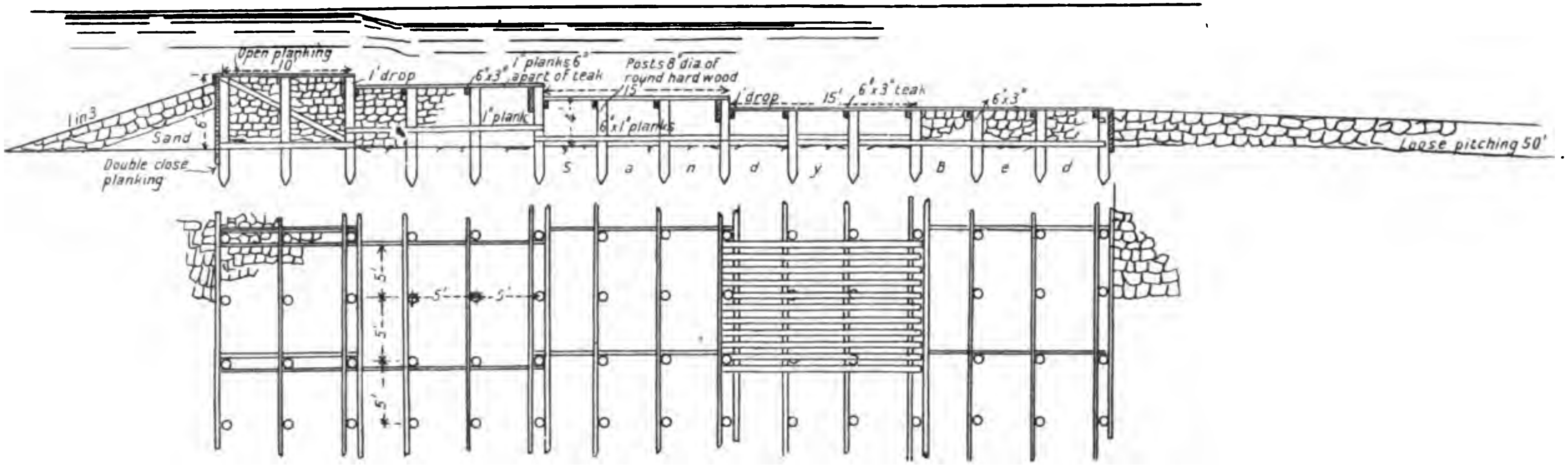


FIG. 16.—Crib Weir (adopted in Burma).

Analysis is as follows:—

| | $A$ | | $\rho$ | | $A\rho$ |
|---|---|---|---|---|---|
| Rear slope . . . . | 20 | × | ·8 | = | 16 |
| Weir wall . . . . | 42 | × | 2 | = | 84 |
| ,, ,, . . . . | 27 | × | 1 | = | 27 |
| Wells . . . . . | 54 | × | ·6 | = | 32 |
| ,, lateral influence . | 36 | × | ·6 | = | 22 |
| Total . . | | | | | 181 ÷ 75 = 2·4 feet. |

What we want here is a floor $4\frac{1}{2}$ feet thick, to counteract the bare head of 7 feet, or, if $H_1$ is considered as 9 feet, one $7\frac{1}{2}$ feet thick, whereas the thickness is only 3 feet. This work also must be founded on clay, or clay and boulders. In all the foregoing the stanchness of the rear apron is assumed.

(72) Fig. 16 represents a section of a crib weir constructed by the author in Burma. The height is 6 feet, and the contents about 400 square feet per foot run. Multiplying this by 1·3, the specific gravity of pitching, we have $A\rho = 520$. This divided by $75 = 6{\cdot}93$ feet, which would just neutralise the bare head of 6 feet. The surface was not packed with large stones, for which there was no time or skilled labour available, but the pitching was prevented from being washed out by covering boards. Eventually, when the weir body is thoroughly grouted with silt and finally settled down, a layer of rubble masonry will take the place of this temporary covering. The value of $l$ works out to 120 feet $= 15H$ ($H$ taken as 8 feet).

Several of these weirs were built, and as long as the fore slope was retained at 1 in 15 they answered perfectly. When this, however, was increased to 1 in 10, the weir leaked from springs underneath and much settlement occurred, showing that the frictional stability was defective. This fact is corroborative of the correctness of the general theory and of the coefficient adopted.

(73) A remarkable example of a weir body constructed mainly of clay on a sand foundation is given in Fig. 17, of the section of the Sidhnai Needle Weir, where it crosses a deep depression in the river.

The profile belongs to type D. The overfall is divided into sections by cross walls 5 feet thick, at intervals of 20 feet, which are spanned by a beam of wood, on which the needles abut. These, as well as the breast wall, are supported on piles planked over. The whole body of the apron and rear slope is formed of clay filling, which is covered by slab pitching 1 foot deep, resting on 3 inches of quarry chips, and terminating in a line of heavy concrete blocks. Presuming the whole as submerged, the head will be 7 feet. The value of $A\rho$ for the weir body is estimated at 559. To this must be added the proportion per foot run of the piers, *i.e.*, $\frac{330}{5} = 66 \times 1\frac{1}{4} = 82$, and some value for the piling which will be taken as half that of a curtain wall of the same size, viz., $\frac{1}{2} \times 100 \times {\cdot}5 = 25$. The sum of these three items will be $559 + 82 + 25 = 666$. This divided by the head 7 feet will give 95 as the value for *C*, *i.e.*, close upon 100, which is the coefficient for this class of river sand.

In places where this weir rests on a clay foundation, 2 feet of concrete is substituted for the piling, and the piers have separate foundations, both quite shallow.

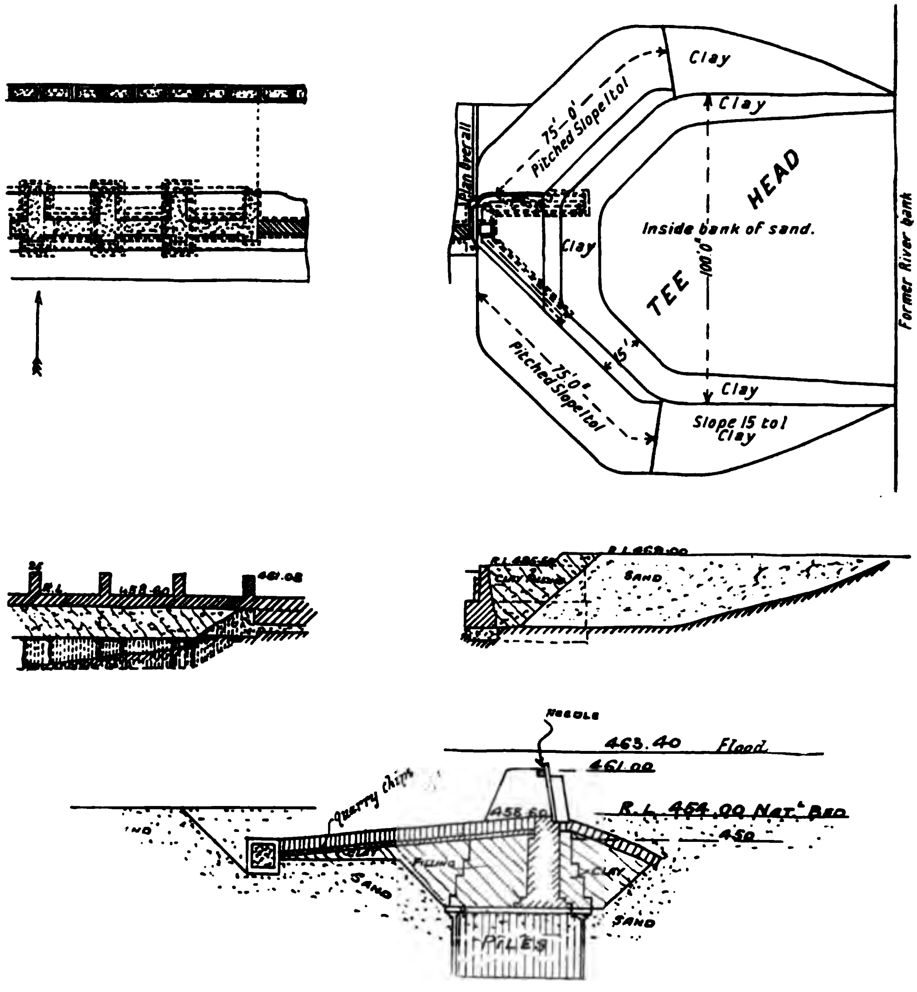
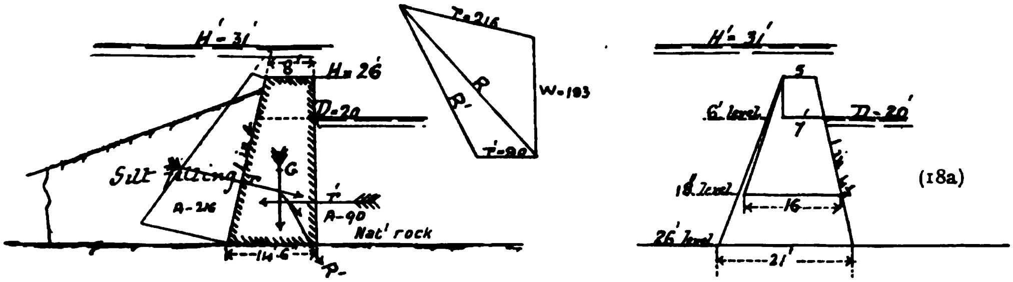


FIG. 17.—Sidhnai Canal Needle Weir, Sutlej River.

(**74**) There is no doubt that where stone is scarce, clay filling or puddle could be substituted with advantage where properly protected. A layer of clay puddle is necessary in the pitched rear slope or in anything beyond it in order to secure its stanchness to percolation; otherwise the transference of the incidence of the head from the weir wall backwards to the commencement of this layer of material, which is essential to ensure its inclusion in the main weir body, cannot be depended on. Puddle placed upstream of the weir wall is quite as effective as masonry if of equal specific gravity, and the same applies downstream as in the case of type D.

**(75)** In Figs. 18 and 19 are sections of weirs built on rock at the head-works of the Karnúl Canals in Madras. When weirs are built on rock, they are necessarily not of the same depth throughout; in such cases, unless a wide roadway is required along the crest for use at low water when the weir is not acting, the top width, if uniform, should be made as narrow as practicable, so as not to waste material in the shallow sections. The fore slope or batter should be uniform, but the back slope should vary so that the base is neither more nor less than $\frac{H_2}{\sqrt{p}}$. The Sankêsela Weir section (Fig. 18) is most unscientifically designed. This weir is 4,500 feet in length, and varies in depth from 6 feet to 26 feet, the general average being 18 feet, but one profile only is adopted for all depths, the face vertical and the back slope one four, in the top width 8 feet. The section is too strong for any depth less than about 15 feet, and too weak for any greater depth.



FIGS. 18, 18a.—Sankêsela Weir.

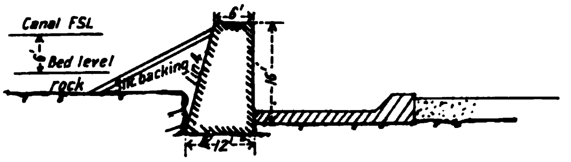


FIG. 19.—Jatur Weir.

Under the assumption that the depth of film, *d*, is 5 feet, and the depth *D*, in the channel below, 20 feet, the resultant line of pressure has been drawn in the profile. Its incidence on the base is just within the outer toe. The wall depends for its stability on the earth backing, and is in fact of the "hybrid" type, *i.e.*, a surcharged retaining wall suitable only for canal falls.

**(76)** If the wall were properly designed for hydrostatic pressure with a base varying as $\frac{H_1}{\sqrt{p}}$ or $\frac{H + d + h}{\sqrt{p}}$ it would cost but very little more than the section in Fig. 18. This has been done in Fig. 18a, three sections being drawn at depths of 6, 18 and 26 feet. The 18 feet depth is taken as a guide for the top width and the fore slope. The width of crest is thus made 5 feet or $\sqrt{H_1}$, $H_1$ being $= 18 + 5 = 23$ feet.

The various base widths are made $\frac{H_1}{\sqrt{p}}$, which is sufficiently near for the purpose. Thus at the deepest part $H_1 = 31$ and $b = 31 \times \frac{2}{3} = 21$ feet nearly. At the 18 feet depth $H_1 = H + d = 18 + 5 = 23$, and $b = 23 \times \frac{2}{3} = 15\frac{1}{3}$, made 16 feet. At the 6 feet depth $H + d = 11$ feet, and the required base width would be $11 \times \frac{2}{3} = 7$ feet.

From the base width due to 18 feet depth thus obtained, we deduct the crest width 5 feet, leaving 11 feet as the combined width of the fore and back batters. Of this we will put $\frac{2}{5}$ in front of the crest and $\frac{3}{5}$ behind, or roughly, 4 feet in front and 7 behind. This gives a face batter of one in four and a half, which will be uniform throughout. The back batter should vary in consonance with the varying base width until the vertical is reached; the back of the wall will then have a wind in the batter, which, however, is quite as easy to build as a uniform batter (*vide* remarks on retaining walls similarly circumstanced, in Chap. I.).

The comparative areas at the three depths are stated below:—

| Depth below Crest. | Areas. | |
|---|---|---|
| | Original. | Revised. |
| 6 | 51 | 34 |
| 18 | 180 | 189 |
| 26 | 290 | 338 |

It is obviously risky to construct an important weir 4,500 feet long, of a section whose stability is contingent on the impermeability of the earth backing. The "hybrid" type is suited only for canals of moderate width, and where the puddling at the back can be closely supervised.

The section (Fig. 19) of the Jatur Weir is likewise of the "hybrid" type, founded on rock.

(77) The thickness of floors or aprons in direct overfalls, *i.e.*, in types B and C, has already been treated of in par. 35, Chap. IV., the formula being $\sqrt{H_1} + 1\frac{1}{2}$. In this $H_1$ is the maximum statical head, *i.e.*, the vertical height of the crest of shutters above the floor level; for type C, $H_2$ is of less value by the thickness of the floor than $H_1$ in type B. The thickness and also length of floors in type C are consequently less than in type B. Where no crest shutters are adopted, 2 or 3 feet, as may be deemed advisable, is added to the actual height of weir above the crest. ($H_2$ is the reduced head in Class C.)

The length of these floors should also naturally be in some proportion to the depth and velocity of the current. These are in some measure dependent on the height of the weir above base of structure, *i.e.*, normal bed or low water levels; consequently as a general rule the length of the floor will be made a definite proportion of $H_1$ and of $H_2$. The usual proportion in canal falls is $2H_1$, and taking Narora Weir as model, a length of $3H_1$ or of $3H_2$ will be adopted. These dimensions have been adhered to in all the alternative designs of weir sections of types B and C already given.

(78) The thickness of $\sqrt{H_1} + 1\frac{1}{2}$ or $\sqrt{H_2} + 1\frac{1}{2}$ should be considered as a minimum value. When the specific gravity of the floor material falls much below $2\frac{1}{4}$, particularly in the case of river beds of high coefficients of 100 or 125, this minimum thickness has to be increased.

An example of this has been given in Fig. 3a, a design for the Narora Weir, where the requirements of frictional stability necessitate increasing the thickness above the minimum. This increase is best effected as shown in the section by retaining the minimum thickness $\sqrt{H_1} + 1\frac{1}{2}$ at the termination of the floor, while at the toe of the weir the thickness has been considerably augmented, in this particular case by one-half.

The wedge-shaped trapezoid thus formed is the most scientific outline for a floor subjected to the pressure of a head of water, which pressure is gradually diminished by the weight of the floor itself.

Thus we see that the thickness given by the formula is subject to revision by the application of analysis for frictional stability.

**(79)** We now come to the talus, or that portion of the floor or apron beyond the termination of the masonry floor, which is composed of stone pitching. The thickness of this (subject also to the test of upward pressure) should lie between 6 feet as a maximum and 4 feet as a minimum. As regards the length of this portion, Narora will be taken as model. The length of its pitched floor is 100 feet or $10H$. Thus for type B, class I., the length of the whole floor, measured from the toe of the weir wall, will be $3H_1 + 10H$. For rivers of class II., *i.e.*, with coefficient 75, this length can be reduced by $2H$. It will then stand at $3H_1 + 8H$.

**(80)** With regard to type C. Owing to the more congested state of the waterway, the talus will require to be longer than in types B or D, and it is proposed to increase the proportion by a length equal to $H$. Thus for type C, class I., the length will be (measured from toe of weir wall) $3H_1 + 11H$; for type C, class II., $3H_1 + 9H$. ($H_1$ in Class C is head to base of floor.)

**(81)** With regard to rivers of class III., the coefficient of which is 125, the formula for types B and D will be $3H_1 + 11\frac{1}{2}H$. In the only example of this class, viz., Fig. 4a of Damietta Weir, the length given to the floor is $(3 \times 12) + (11\frac{1}{2} \times 10) = 36 + 115 = 151$, made 150 feet.

**(82)** Other examples are: Fig. 2a, type C, class I. Length given is $(3 \times 13) + (11 \times 10) = 40 + 110 = 150$.

In Fig. 2b, type D, class I., the length is $(3 \times 13) + 10 \times 10 = 140$.

In Fig. 5a, Jamrao Weir, type D, class I., the length given is $(3 \times 8) + (10 \times 8) = 122$. This is an exceptional case. Owing to the lowness of the weir, the latter being only one-half of the head, and the thickness of the material used, $H$ is taken to be equal to $H_1$, the formula being resolved into $L = 13H_1$.

If the formula $3H_1 + 10H$ were used, the length would be only $24 + 40 = 64$ feet, which is insufficient even for frictional stability.

In Fig. 7b, Jobra Weir, $H_1$, measured above normal bed level, is $13\frac{1}{4}$ feet, while $H = 10\frac{1}{4}$ feet, whence $L = (3 \times 13\frac{1}{4}) + (9 \times 10\frac{1}{4}) = 40 + 92 = 132$ feet.

With regard to type A, the length below breast wall, taking Okhla as model for rivers class I., coefficient 100, would be $21H$. For class II., coefficient 75, $16H$.

(**83**) These rules are embodied in the Table given below.

| Class. | River Coefficient. | Type. | Distance of End of Talus below Toe of Weir Wall. |
|---|---|---|---|
| I. | 100 | B and D | $3H_1 + 10H$ |
| I. | 100 | C | $3H_1 + 11H$ |
| I. | 100 | A | $21H$ |
| II. | 75 | B and D | $3H_1 + 8H$ |
| II. | 75 | C | $3H_1 + 9H$ |
| II. | 75 | A | $16H$ |
| III. | 125 | B and D | $3H_1 + 11\frac{1}{2}H$ |

Type A is not given for class III., as the anicut type is most unsuited for rivers of so high a coefficient. In fact, this type in any class of river is deemed quite obsolete.

These rules for thickness and length are intended, like all other empirical formulas given in this book, as *guides* to design. They can easily be modified to suit special circumstances or ideas. The value of the co-factor $l$ (paras. 19—21) has not been taken up in the calculations for frictional stability by reason of its being in all cases in excess of strict requirement.

(**84**) In the foregoing, frequent reference has been made to channel bar steel sheet piling and its superiority in point of cost and time employed in erection to the cumbrous and antiquated Indian system of curtain wells or blocks. Further, it has been conclusively proved that, except in weirs on rivers of class I., rear curtains of sheet piling are all that is required, and those of moderate depth. If the talus is not to be absolutely depended on, a fore curtain of sheet piling at the termination of the masonry floor is a desideratum, though seldom required for purposes of frictional stability.

(**85**) Steel interlocking channel bar sheet piling has been used for some years in the United States with great success for deep coffer dams and other purposes where water has to be excluded from foundations. Owing to the width of this sheet piling, it is capable of withstanding considerable lateral pressure. It can be used in a straight line, or to form almost any shaped enclosure. Its very simple construction is shown in Fig. 20.

The channel bars are driven with the greatest ease by an ordinary pile driver. They can also be drawn without difficulty, and used over and over again for temporary work. They become absolutely watertight in a short time automatically, by particles being forced into the interstices, or if necessary, this result can be effected by throwing rubbish into the external water.

(86) Being extremely portable and comparatively light, they are eminently suited for countries where long railway and also cart conveyance has to be employed. They are in every way superior to the heavy cast iron pattern used in the Assiût Regulators, and do not require to be in short lengths joined up, being manufactured, if so required, 30 or 40 feet in length.

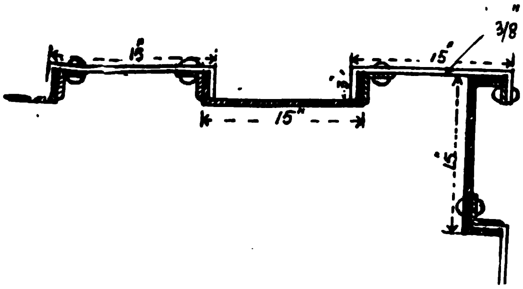


FIG. 20.

The patent belongs to the British Steel Piling Co., of Dock House, Billiter Street, London, E.C., but whether the protection of a patent of this kind is recognised in India is not known. If not, it would be a very simple matter to manufacture them in any large shops. For sheet piling for short depths, as 10 or 15 feet, the ordinary iron channel bars of commerce would answer perfectly well, and their cost should not exceed some £6 or £7 a ton delivered at an Indian port.

Another type, the bulb and cup pattern, already mentioned in para. 33, would seem more suitable where a simple watertight barrier is required, not subjected to pressure. This section is given in Fig. 21. The stanching of the joints in this pattern can be easily accomplished by first pumping out the sand in the hollow and then filling the space up with cement grout under pressure.

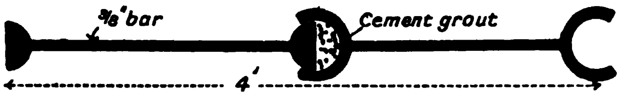


FIG. 21.

Where sheet piling is liable to pressure on one side, as is possibly the case in fore sheet piling, reinforced concrete piles of a wide pattern, suitable for driving into sand only, would probably be the best type to adopt. They would possess greater weight and solidity than the thin plain sheet piling, in addition to greater strength and durability.

(87) Before concluding this chapter some notice is required of the flank walls of a weir. With one canal only taking off, the weir proper is bounded by a dwarf flank wall dividing the weir wall, floor and talus from the undersluice channel on one side, and on the other a long high flank wall upholding the termination earthen afflux bank. Both walls require deep foundations. The flank wall should have a considerable face batter, say one in four or five, and consequently can be of light section. The foundations require to be well stepped forward so that the line of pressure passes within the final base.

The Indian undersunk blocks or wells of brickwork are very suitable for the wall foundation, as much of it is not wanted, and the tedious process of construction would not delay the rest of the work, as miles of curtain wells certainly do.

# CHAPTER VII.

## CANAL HEAD WORKS.

### PART II. UNDERSLUICES.

(1) IN all weirs constructed across wide rivers having sandy beds, Undersluices, or, as they are sometimes termed, scouring sluices, are a necessary evil, the expense of these works being generally very great.

The function of Undersluices is twofold: First, to train the deep channel of the river, during low water, past the canal Head. This is a very necessary duty in order to ensure a constant supply to the canal at a time when there is very little water in the river. Secondly, as a preventive measure, to stop or palliate the entrance and deposit of heavy sand into the canal during flood, and to keep a clear deep channel in front of the canal Head when the river is low. For both purposes the scouring action of these sluices should be powerful.

(2) The floor or sill of Undersluices is almost invariably placed at the average river bed level, or else at low water level, which may be a foot or two lower. Thus the maximum statical head to which work is subjected will be the height of the weir plus that of the crest shutters if the latter are adopted.

The ventage given to Undersluices is generally some proportion of that of the canal Head Regulator, though it would be reasonable to suppose that it should bear some definite ratio to the width of the river or to the obstruction offered by the weir.

(3) As a matter of fact the efficiency of Undersluices as regards the prevention of silt deposit depends more on the relative levels of the sills of the canal Head Regulator and of the Undersluices, than on the ventage given. By reference to the Table of Statistics given later, it will be seen that the relative ventage of the Undersluices and the canal Head varies roughly from one to one to two to one.

The former proportion appears most in favour, and it might be assumed that the ventage of Undersluices should not be less than that of the canal Head, and may be more, should special circumstances in connection with the river seem to demand a greater ventage.

(4) It need hardly be stated that the discharge through an Undersluice is naturally immensely in excess of that of the canal.

As the object of an Undersluice is to pass water at a high velocity in order to scour out deposit for some distance to the rear of the work, it is evident that the openings should be wide with as few obstructions as possible in the way of piers, and should be open at the surface, the arches and platform being built clear of the flood level. Further, in order to take full advantage of the scouring power of the current, which is at a maximum at the sluice itself, diminishing in velocity with the distance to the rear of the work, it is absolutely necessary not only to place the canal Head as close as possible to the Undersluices, but to recess the Head as little as practicable behind the face line of the abutment of the end sluice opening.

In the examples of old Indian works, which will now be given, we shall find that these conditions have often been violated with great detriment to efficiency. This in part was unavoidable, as prior to the introduction of iron drop gates fitted with anti-friction rollers, the openings of Undersluices and Regulators generally had perforce to be designed of narrow width, in order to admit of the gates being worked under a pressure of water. Thus we see that the width of waterway in each opening was limited to 4, 5 or 6 feet only, whereas now such openings vary from 10 feet to 25 feet in width.

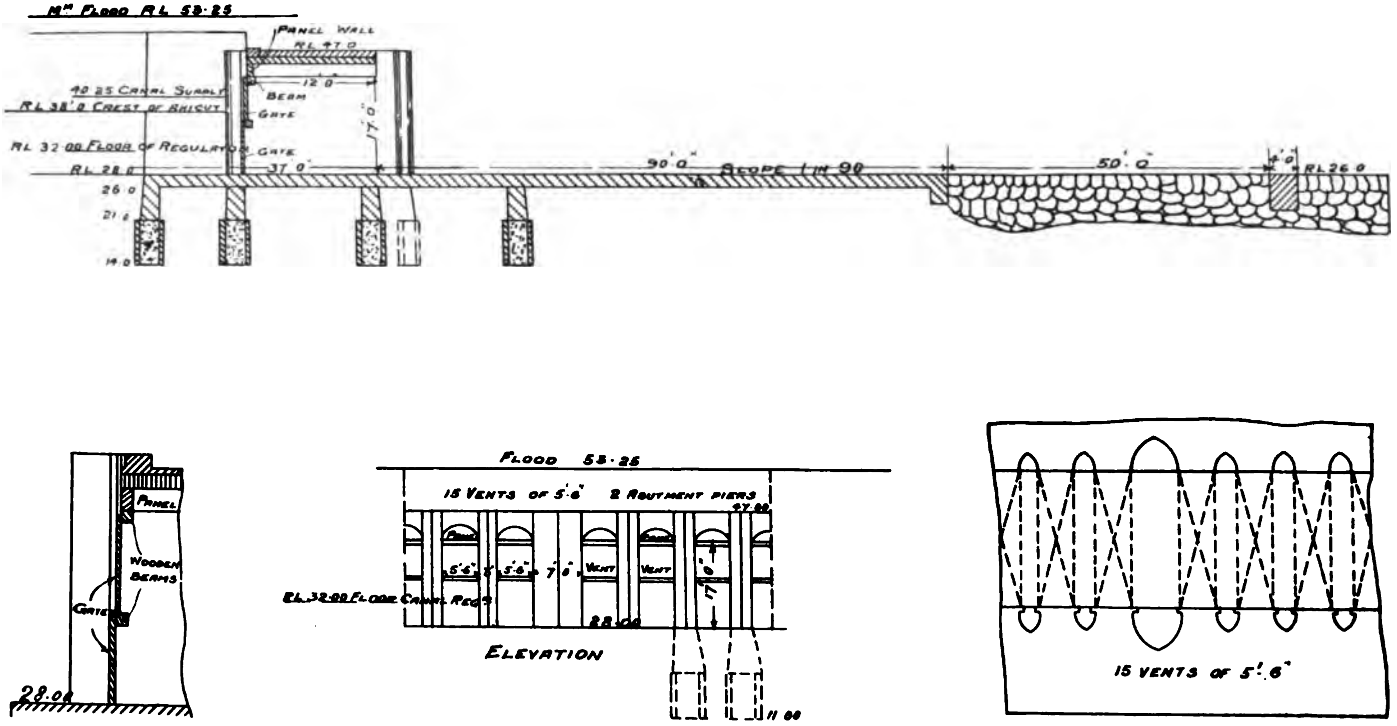
(5) With regard to water pressure, Undersluices perform a *rôle* identical with that of river regulators, or, as they are also termed, open dams or barrages; that is to say, they only require to be designed for partial or limited regulation. They are not intended to hold up the full flood depth of water in the river, but as a rule only when the river is low are the gates closed, in order to dam up the water to a certain level, sufficient to force supply down the adjacent canal Head. In some cases, where the velocity of the current at full flood is so great as to overstrain the flooring, the lower sluice gate or both gates are kept down, acting then as a submerged weir.

When the river rises much above canal full supply level, no regulation is required at all, and the gates then can be lifted clear of the flood to afford a free passage to the current.

Canal Head Regulators, on the other hand, have occasion to be entirely closed during the highest floods in the river, consequently the regulation they have to perform is entire, not partial, so that these works are subjected to a much greater statical stress than Undersluices, and consequently, for convenience of manipulation, are usually designed with narrower openings than are necessary or desirable in the latter.

(6) Fig. 1 is a representation of the Dauleshwiram Undersluices of the weir of that name on the Godaveri River in the Madras Presidency.

This old work consists of fifteen vents of 5½ feet width, which are arched over above canal supply level, presenting a level platform 19 feet high above sill. The openings are completely closed by wooden draw gates in grooves, reaching above the spring line, while the arch segments are closed by narrow panel walls which are supported on wooden beams. The upper gates rest on another line of wooden sills against which the top side of the lower gates



(1c) (1a) (1b)

FIGS. 1, 1a, 1b, 1c.—Dauleshwiram Undersluices.

bears. The maximum flood line is 25¼ feet above the sluice sills, and so the whole work is submerged during high floods, a depth of 6¼ feet passing unobstructed over the platform.

| | |
|---|---|
| The R. L. of the sill is | 28·00 |
| „ „ of the sill of head regulator | 32·00 |
| „ „ of the crest of the anicut | 38·00 |
| Full supply level in canal | 40·25 |
| Level of submerged platform | 47·00 |
| Level of maximum flood | 53·25 |

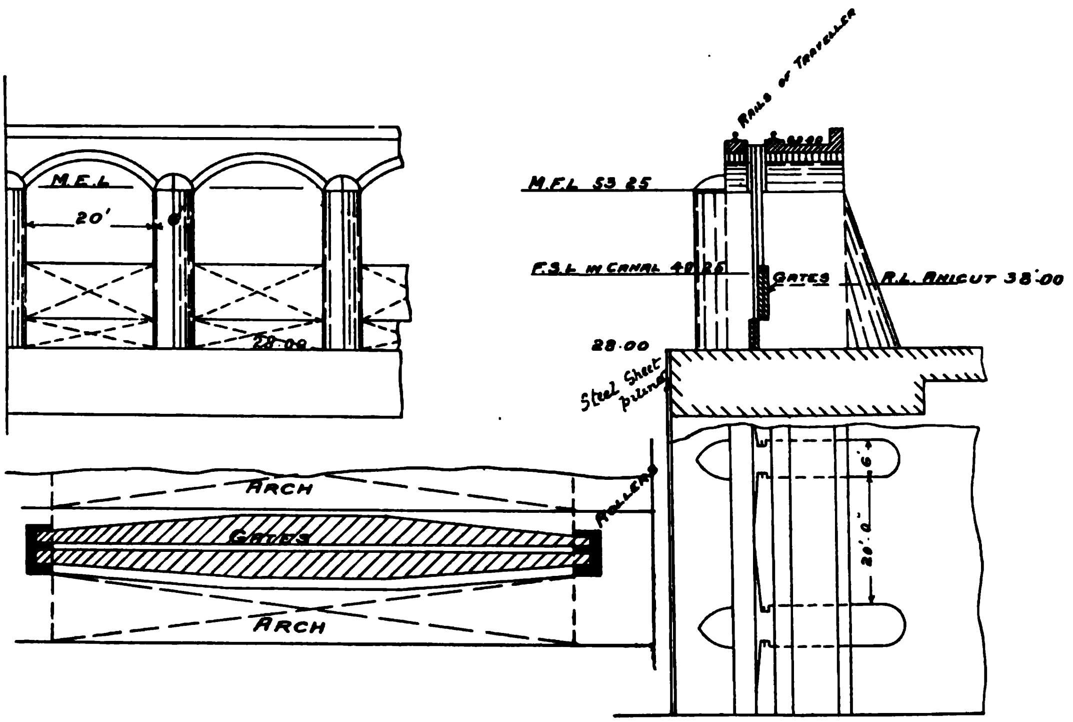


FIG. 2.

On the occasion of floods the gates are either suspended at the top of the grooves, in which position they will still somewhat obstruct the waterway through the sluice openings, or else are taken out bodily and removed.

(7) The obstruction offered in this old work to the free passage of floods is very great. Regulation is only required up to a depth of 12 or 13 feet, so that if the work were reconstructed on modern lines it would be less expensive and much more effective. The remodelled design would roughly consist of the following:—Four spans of 20 feet with piers 6 feet thick, double or treble iron gates 13 feet high, running in separate grooves with anti-friction rollers. Spring of arches to be at R. L. 53, or 28 feet above sill; the platform at top 7 feet higher, or at R. L. 60·00. This will allow the gates to be drawn up and left hanging clear of the flood line. The gate

grooves will be situated outside the main arches, as in the original work. To facilitate the lifting of the gates, another arch and narrow platform is desirable outside the grooves. Its function is to carry the outer rail of the travelling winch, the inner resting on the edge of the main platform.

This latter need not be over 10 or 12 feet wide, the pier ends battering outwards, in order to spread the weight of the superstructure and to afford length of base sufficient to bring the resultant line of pressure within its middle third.

The work thus remodelled is shown in Fig. 2. The design is very similar to that of the Assiût Regulator, plan of which is given in Fig. 7.

(8) The next plan to be examined is that of Fig. 3, of the Sangam Undersluices on the Penner River, Madras Presidency. This is a much more modern work than that at Dauleshwiram, having been constructed in 1870—80.

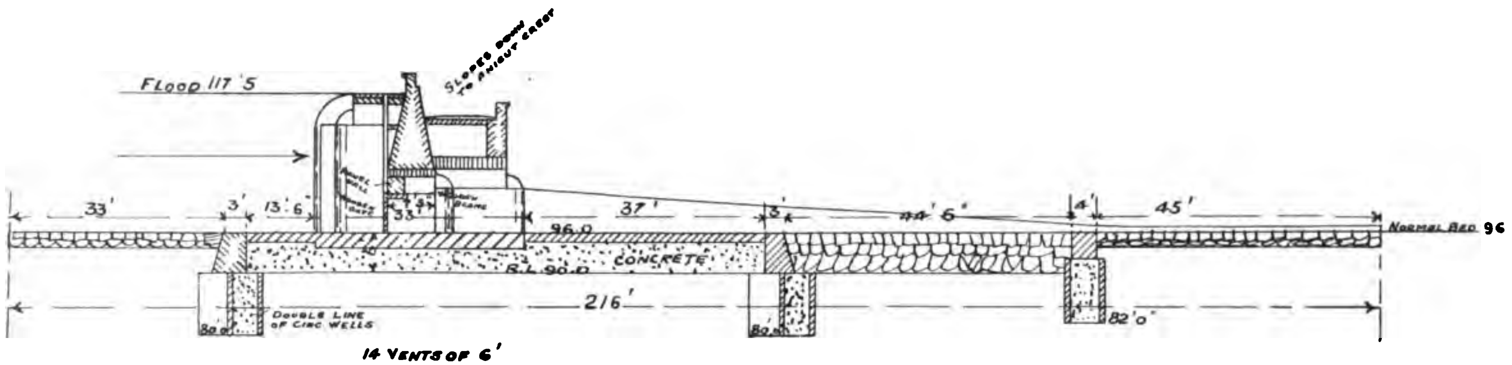
This design is on the principle of a Reservoir escape sluice, or canal Head, for which purpose, with certain modifications, it would answer well; but it cannot be said to be in the least adapted for the *rôle* of a scouring sluice. As will be seen by reference to the plan, the design consists of a series of small sluice ways 6 feet wide and 5 feet high, which are topped by a heavy breastwall carried up to above flood level, which barrier effectually prevents any passage of water except through the sluice vents. This system is the very reverse of what is now deemed correct, which latter principle, as has already been explained, is to afford as free a passage as possible through the work, so as to cause effectual scour of deposit in front of the canal head at the rear.

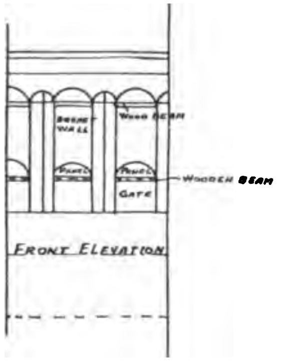
The design, if remodelled on these lines, would be very similar in appearance to Fig. 2, or to the Assiût Regulator and Rupar Undersluices, viz., a simple bridge of a few large spans of 15 feet or 20 feet with arches and platform above M. F. L.

A noticeable peculiarity in Fig. 3 is the roadway provided behind the breastwall, which is carried by a separate set of arches springing from every alternate pier, which pier is lengthened to receive them. This roadway slopes down to the anicut crest, and is probably used for cart traffic.

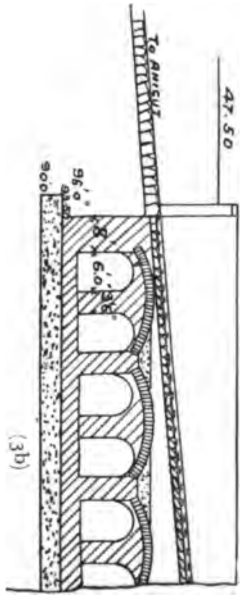
The defences of the floor against erosion or upward statical pressure are very considerable. There are triple lines of interlacing curtain wells, one at either end of the masonry apron and the third at the termination of a length of very deep rubble pitching. One minor point deserves mention, and that is the disposition of the dwarf partition wall dividing the Undersluice floor from the weir. This wall is given an outward trend, so it encroaches on the weir apron and talus. This cannot be considered a good arrangement; the partition wall should be perfectly straight and normal to the weir crest, as shown in the block plan of the Son Weir (Fig. 6c). If widening of the exit waterway is deemed requisite, this should be effected on the opposite side by throwing the flank wall inwards.

(9) The following remarks will serve to clear up a somewhat obscure point already alluded to in para. 10, Chap. III.

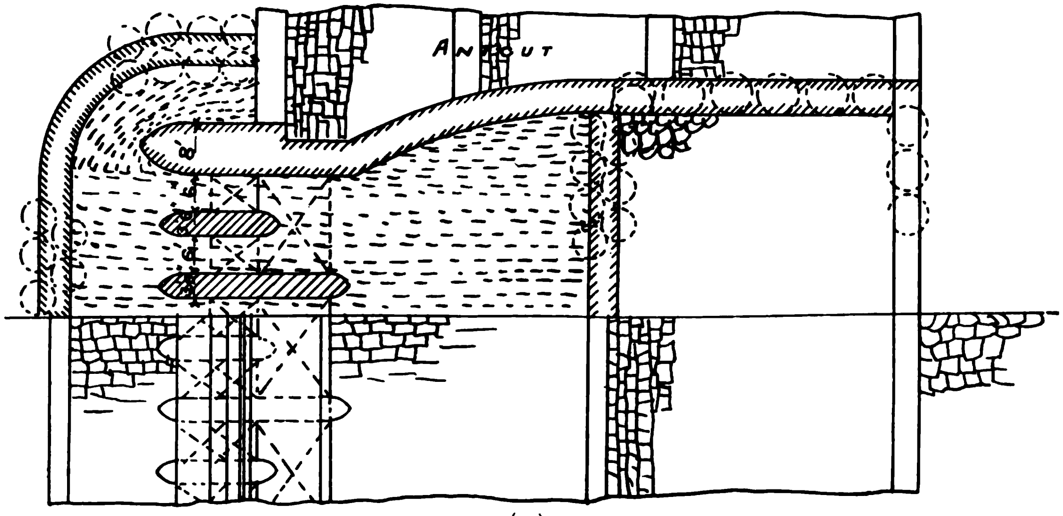




(3c)



(3b)



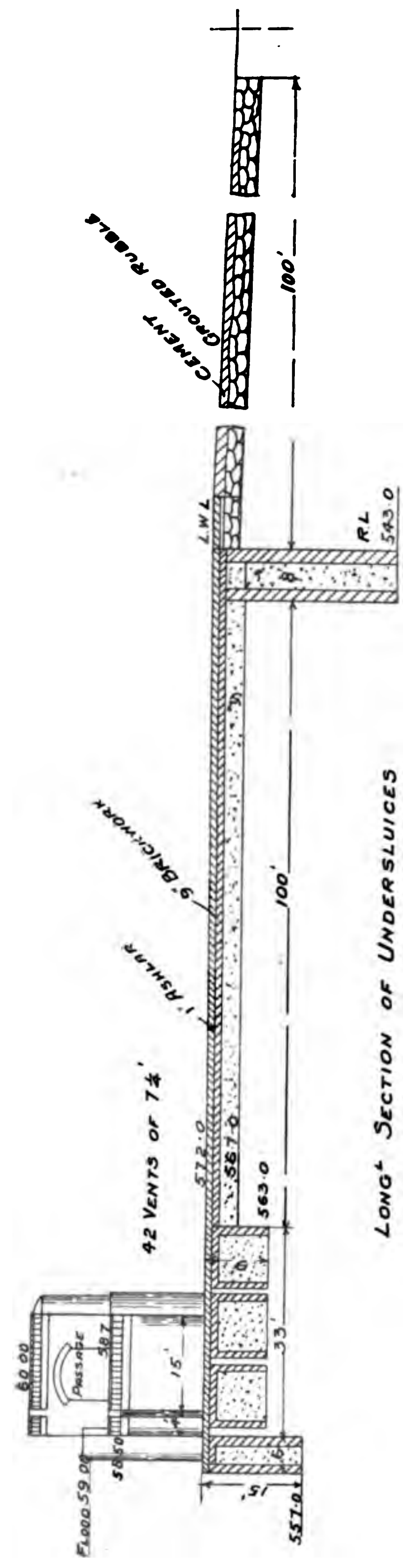
(3a)

FIGS. 3, 3a, 3b, 3c.—Sangam Undersluices.

The maximum horizontal stress on the superstructure of a partial Regulator may possibly be in excess of what is produced by the maximum statical head, which means that a greater effect can be produced by a less head. This is owing to the influence of the increased depth of the water fore and aft of the work. We have already seen in Chap. III. that the same applies to weir walls, particularly when subject to flotation. The opposing horizontal overturning moments are proportionate to the cubes of the depths of water on either side of the dividing wall or structure, if the division is complete, which, however, is not quite the case. Thus, supposing the maximum statical head on an Undersluice Headwork to be 10 feet; this must occur when there is no tail water, *i.e.*, when the whole river low supply is taken into the canal. The overturning moment $M$ can then be represented by $10^3 - 0^3 = 1{,}000$. Now with 2 feet passing over the weir and gates, the depth of water downstream can be calculated with sufficient accuracy for the purpose by using Table II., Chap. III. Assuming the river bed slope to be 1 in 10,000, the ratio of rise will be about ·35. Then $D = \frac{d}{r^{o}} = \frac{2}{{\cdot}35} = 5{\cdot}7$ feet. The depth upstream will then be 12 feet, and downstream 5·7, and $M$ will be $(12)^3 - (5{\cdot}7)^3 = 1{,}728 - 185 = 1{,}543$. Thus we see that a much greater horizontal stress is induced by the lesser head of 6·3 feet than by the maximum head, which is 10 feet. This assumes that the greater area of pressure is that of a full triangle, not of a truncated triangle, as is actually the case.

Taking another level, supposing 2·8 feet passes over the weir, then by Table II., $D = 7{\cdot}5$ feet, and $M = (12{\cdot}8)^3 - (7{\cdot}5)^3 = 2{,}097 - 421 = 1{,}675$. With 3½ feet over the weir, in the same way, $M$ will be $(13{\cdot}5)^3 - (9{\cdot}2)^3 = 2{,}460 - 778 = 1{,}682$. This will be about the maximum value of $M$, it being diminished by any further rise of the head water. The head producing this effect is $13{\cdot}5 - 9{\cdot}2 = 2{\cdot}3$ feet only. To allow for this increase in stress a safe approximate rule would be to increase the maximum statical head by about ¼ when calculating the incidence of the resultant line on the base, due to the water pressure on the superstructure, or else to allow a similar margin in the said incidence within the middle third of the base. In this calculation the effect of the tail water is neglected. As regards the transmitted vertical upward pressure on the floor, the maximum net statical head naturally has alone to be considered. This increase will be much the same as that allowed in weirs not provided with crest shutters. Where, however, crest shutters are used, the actual net height of shutter crest above floor should be increased as above indicated. The incidence of the exact maximum stress on the pier bases can be found by trial graphical process, assuming different values for $H_1$, and its reciprocal $D$. In most cases this is hardly necessary, as the Regulating bridge generally possesses considerable excess of stability, with regard to the limited regulation required of it.

(10) The Narora Undersluices are given in Fig. 4. These were built about 1875—80, and present a marked advance on the last example. The vents are 7½ feet wide in groups of three spans, separated by piers 7½ feet thick.

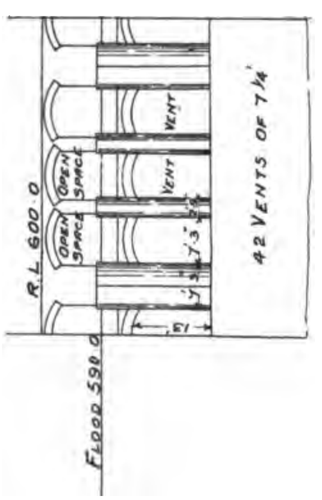


Figs. 4. 4a.—Narora Undersluices.

The intermediate piers are 2·9 inches thick, or ·37$S$ nearly. There are two tiers of arches, through both of which an open slit is provided for the passage of the draw gates. The travelling winch runs on the top platform, which is 28 feet above floor level, a height sufficient to allow the gates, when raised up, to hang clear of the flood line. The lower tier of vaulting forms a lower platform, 3 feet below high flood level, and the space between the piers, which continues up to the upper tier, is open, allowing the passage of water above the lower platform. The upper parts of the piers are pierced by wide openings at right angles (*vide* Fig. 4a), thus forming a vaulted passage, which is of great convenience for the manipulation and stacking of the gates when drawn up.

(11) It is quite evident that this design could be greatly improved by simplification. The spans should be widened, say to 15 feet or 20 feet, with piers 4½ feet to 6 feet thick, the abutment piers being abolished together with the middle platform. The profile would then closely resemble Fig. 2 or the Assiût Regulator (Fig. 7).

The maximum statical head of the floor is 13 feet. The value of $Ap$ for the superstructure is estimated at 526



Front Elevation of U.S. (4a)

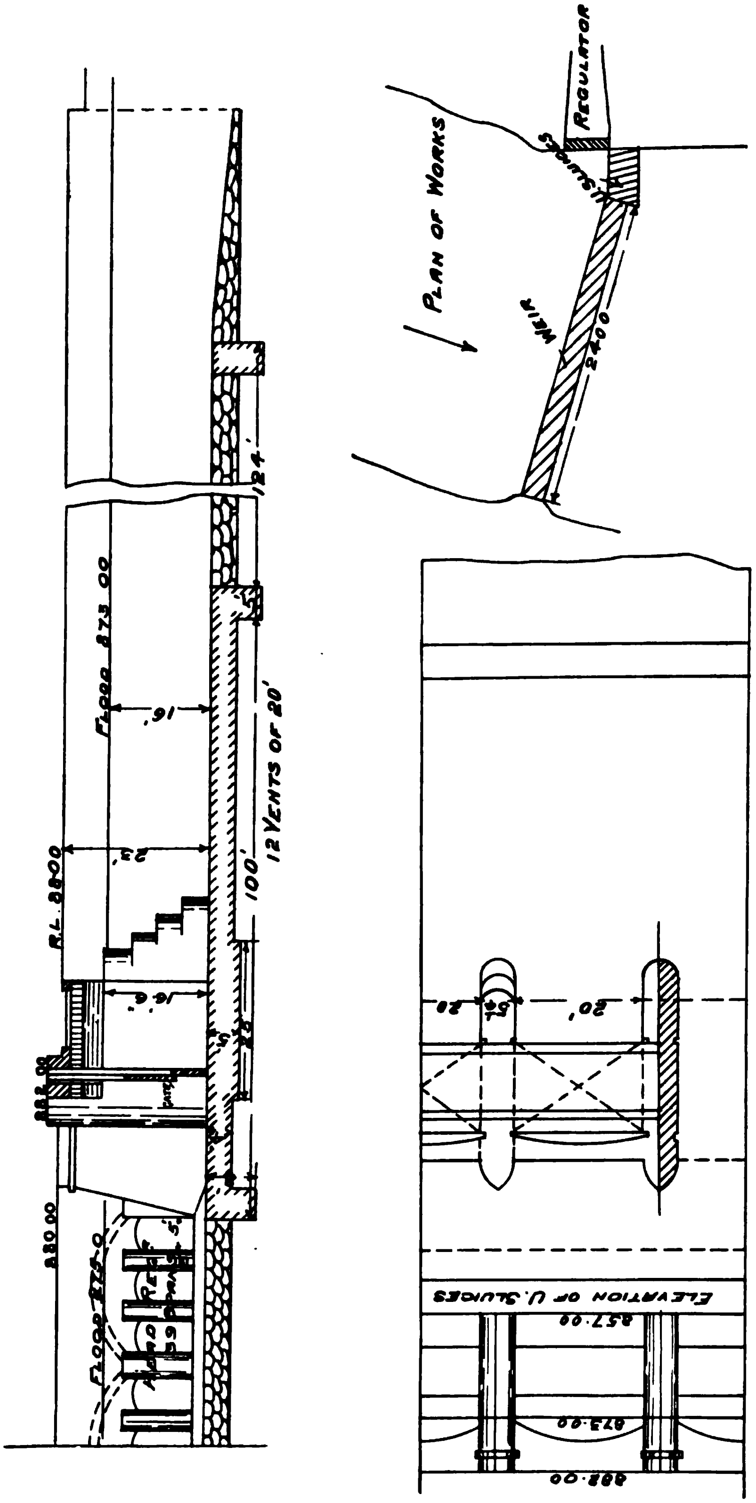
and that of the foundations below at 360; total, 886. Dividing by the proper coefficient 100, the head neutralised at this point is 8·9 feet, leaving 4·1 as upward pressure. This is met by a floor thickness of 5 feet with $(\rho - 1) =$ unity—more than sufficient. The rest of the effective weight of the work is far in excess of the requirements of statical stability. In the case of Undersluices, however, dynamical considerations dominate the required length of floor, which cannot be a matter for calculation, but one in which successful precedent has to be followed. As will be seen later, when the dimensions of floor will be discussed, its length will have to be 10 $H$, or ten times the height of the crest of the weir above the floor.

(12) The Undersluices of the Rupar Weir (Fig. 5), on the Sutlej River, at the head of the Sirhind Canal, are remarkable as being by far the best of any Indian works of this description of which we possess records. Their characteristics are boldness and simplicity. No doubt this design was the precursor of the Assiût Regulators, which are built on closely similar lines. The vents are 20 feet wide bridge openings, the arches springing at maximum flood level. The piers are 5½ feet thick; the floor of the bridge is a solid mass; being founded on clay, or clay and boulders, the thickness can thus be reduced below what would be requisite on a sand foundation.

The superstructure is simple and effective, and consists of a platform carried by a tier of arches springing at flood level, and divided as usual into two parts by the slit for the gates and grooves. The double gates are 12½ feet high, just sufficient for the regulation of the supply in the canal Head, which is at R.L. 865·50, *i.e.*, 11½ feet above the floor of the Undersluices. The floor of the Head Sluice is 2 feet higher than the latter.

(13) We now come to a different type of Undersluice from any of the foregoing, which has only been adopted in the Province of Bengal. The governing principle of this design is to afford a free passage of flood water between and over piers, which are only of the necessary height for regulation purposes, and have no arched superstructure whatever. The water is held up by collapsible gates of large size, working between the piers.

The designers of these open sluice ways were evidently of opinion that, having provided a free passage so different to that hitherto usually adopted, it was unnecessary to have the Undersluice floors at a much, or any lower level than that of the canal Head Regulator. This has proved to have been a fatal mistake, for as matters now stand, the effect of the Undersluices is to accentuate silt deposit in the canal, which is still further encouraged by the recessed position of the Head Regulator. It is absolutely necessary that the difference of floor level of these two adjacent works should be considerable. With the heavy drift sand found in these rivers a difference of 4 feet would be none too much. With the assistance of the lower gate of the Head Sluice, or a dwarf weir wall, this arrangement would admit of a deposit of 6 feet of sand in front of the Regulator and behind the Undersluices, which accumulation could be cleared off in a very short time by opening the Undersluices, and the scoured channel would then be left ready for a further



(5a)

(5b)

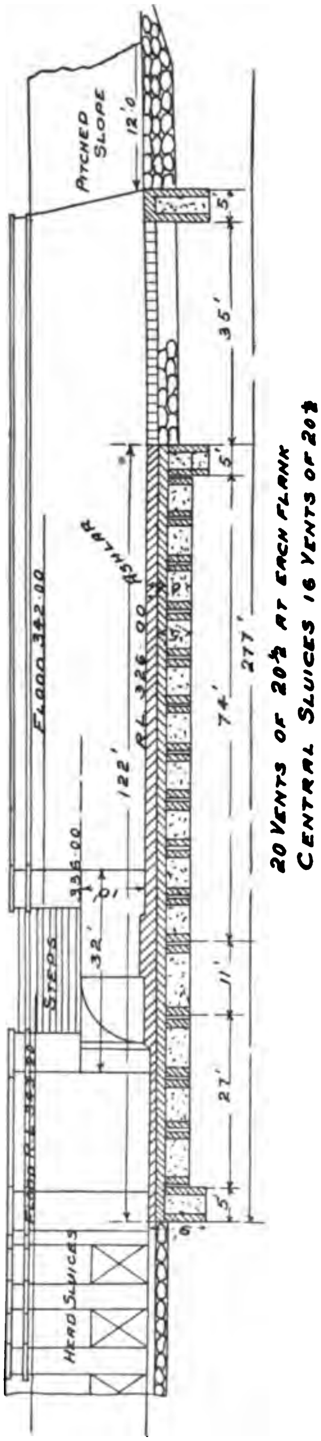
Figs. 5, 5a, 5b.—Rupar Undersluices, Sutlej River.

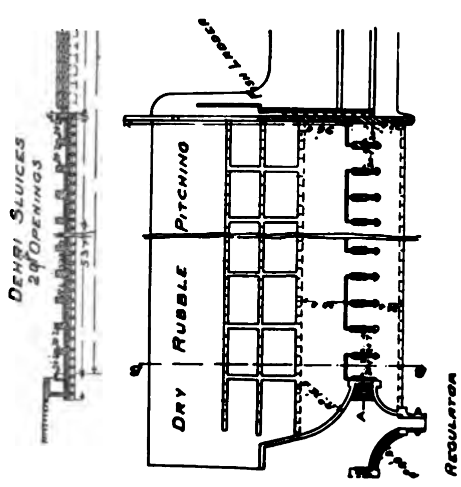
deposit, which will again be swept away by a similar process. By this means, and also by drawing the water over one or two of the gates of the canal Head, and not underneath, heavy deposit can be successfully kept out of the canal. But on the other hand, if the sill levels of the two works are identical, there will be no spare depth available for deposit, and consequently it must be swept into the canal, where its periodical removal forms a heavy annual charge.

(14) Fig. 6 represents the Dehri Undersluice on the Sôn River. This consists of twenty openings, 20½ feet wide, divided by piers 5 feet thick and 33 feet long. These piers are only 10 feet high. The openings are closed by double collapsible wooden gates 20 feet long and 10 feet high. The second gate is for temporary use, and is first raised by hand, which enables the real self-falling gate to be hauled upright by tackle, when the temporary first gate is lowered out of the way.

The falling gate is on the same principle as weir crest shutters, being pivoted at the end of inclined rods. On the gate being overtopped it doubles over and falls automatically, the shock being lessened by a hydraulic brake.

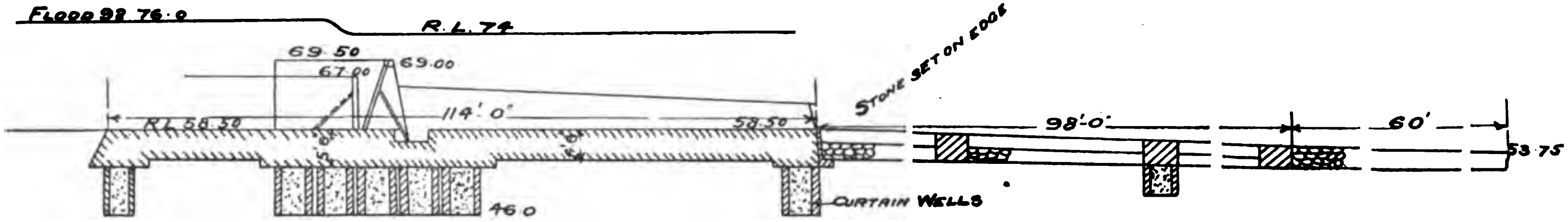
(15) As the flood level is only 7 feet above the top of the piers, it would have been a



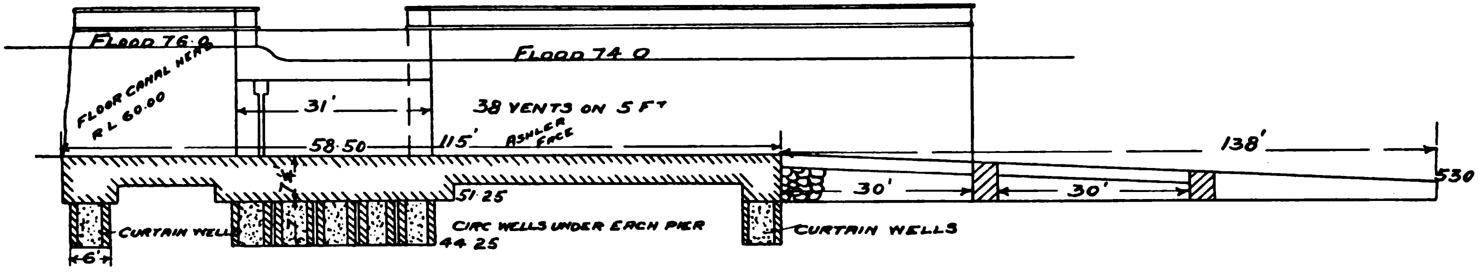


(6a)

Figs. 6, 6a.—Dehri Undersluices, Sôn River.



SECTION THRO' CENTRE SLUICES 10 BAYS OF 45' PIERS 5' THICK

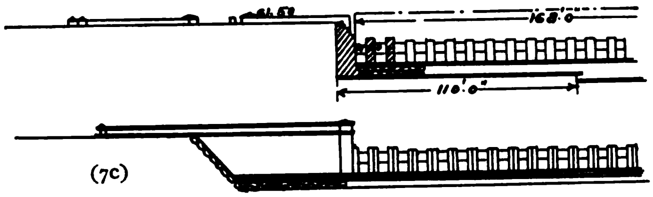


SECTION THRO' UNDERSLUICES BELOW CANAL HEAD 38 VENTS OF 5' PIERS 4' THICK

(7a)

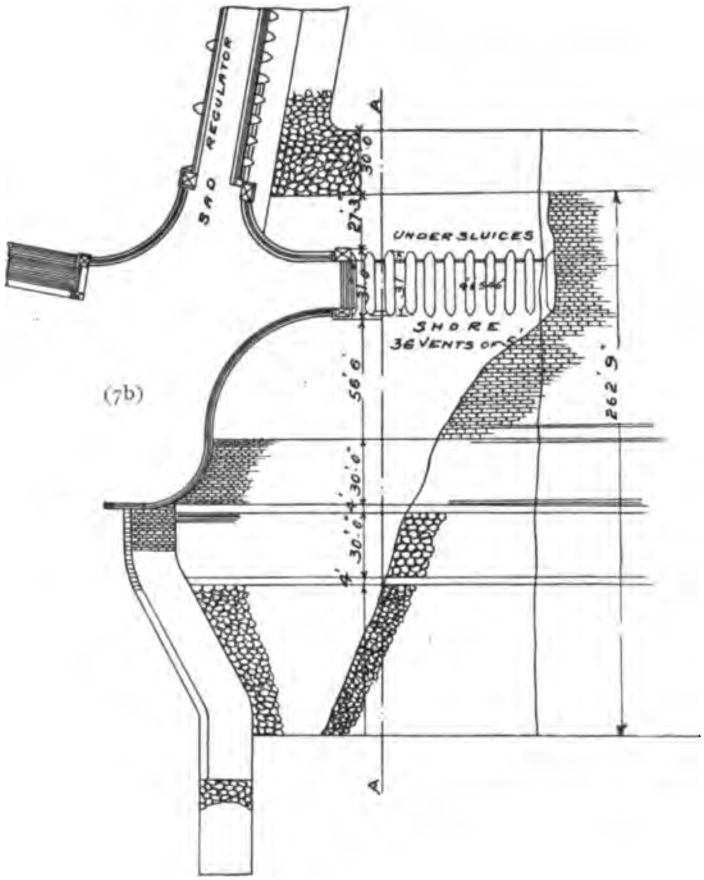
FIGS. 7, 7a.—Jobra Undersluices, Mahanadi River.

much simpler plan to have adopted an overbridge, as at Rupar, with roller draw gates, raised by a traveller. This weir is provided with



(7c)

ELEVATION OF UNDERSLUICE



(7b)

BLOCK PLAN.

FIGS. 7b, 7c.—Jobra Undersluices, Mahanadi River.

sixteen central sluices of 20½ feet width. The object of central sluices is to prevent cross-currents and to train another channel straight on to the weir. These expensive works have, however, proved quite useless for any purpose.

The only practical method of preventing silt islands forming above the weir is to keep the weir crest low and make use of iron collapsible crest shutters.

On the Jamrao Weir, recently constructed, these latter have been made 4 feet high, and a portion of the weir adjoining the Undersluices is made 4 feet lower than the rest, having 8 feet high collapsible shutters. The old Betwa Weir is provided now with 6 feet, and the new with 8 feet shutters.

The block plan (Fig. 6a) shows the position of the canal Head, which is recessed much too far behind the Undersluice abutment, forming a veritable silt trap. The alignment of the dwarf wall dividing the Undersluice and the weir is shown on the block plan. This arrangement of widening the escape channel towards the river bank and not on the side of the weir is correct.

**(16)** The Jobra Weir central sluices on the Mahanadi (Fig. 7) are constructed on the same principle as the ones just described, but they are not self-falling. These gates are no less than 45 feet long and 11 feet high. The Undersluices are formed of isolated piers 4½ feet thick, and only 5 feet apart. They are apparently closed by wooden baulks or shallow draw shutters superimposed. There is no superstructure, so that a temporary plank platform has to be erected and the shutters or baulks removed before the flood season. This arrangement is decidedly primitive.

The masonry floors of both sets of sluices are generally 4½ feet thick, with an additional thickness of 1 foot in the central and 3 feet in the shore sluices; beneath this again is a solid mass of undersunk blocks, 7 feet and 9 feet deep respectively. The length of the masonry floor is 115 feet in both cases, *i.e.*, about $10H$ counting beyond sluice piers.

The central sluice floor failed on one occasion altogether, and its repair cost a quarter of a million rupees.

It is considered that the down-stream or fore-curtain wall should be much deeper, as an insurance against disaster in case of failure of the pitched talus. The sill of the Head Regulator is raised only 1 foot above that of the Undersluices, which is far too little.

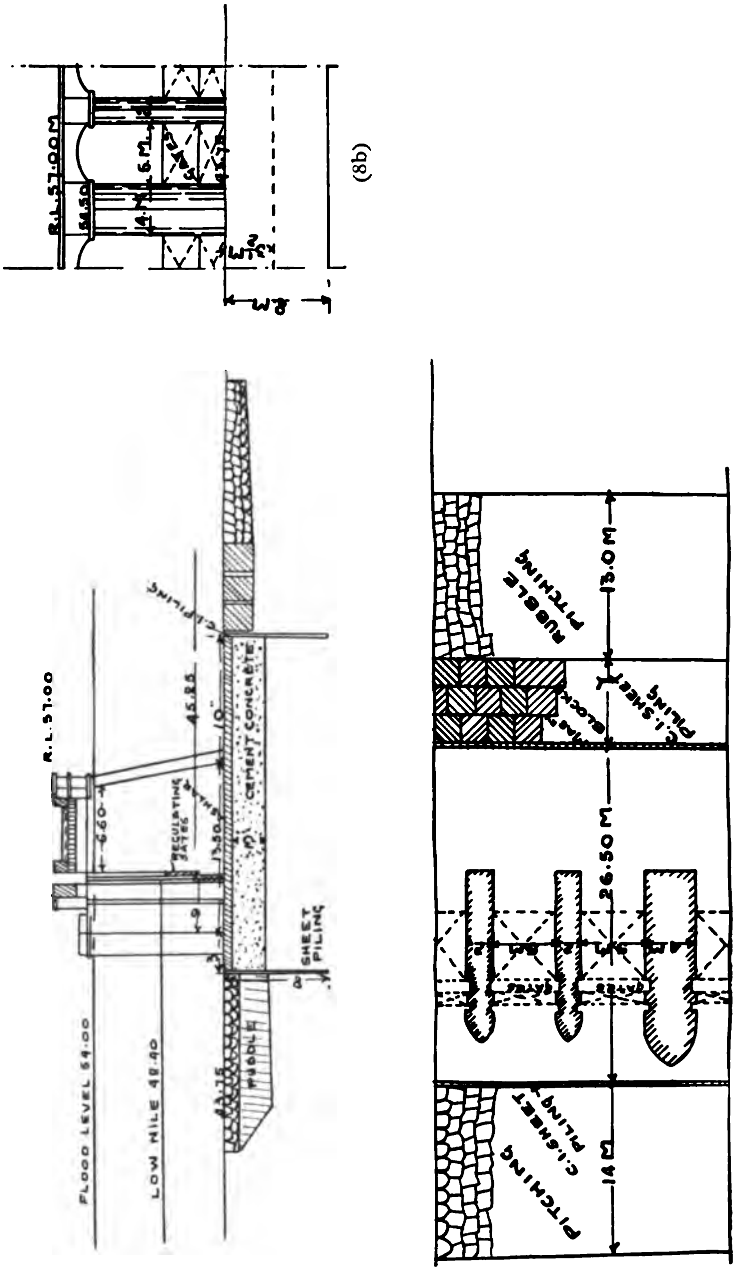
**(17)** The Assiût Works, to which reference has more than once been already made, are the latest exponent of partial regulators.

The plans of this work are given in Fig. 8.

The Assiût Regulator was built across the Nile to regulate the supply in the Ibramiya Canal.

This work will never be entirely closed, and the outside difference of level that will ever be provided against is that between R.L. 48·40 and 45·85, viz., 2·55 metres (*vide* Fig. 8). This is the head, on the extent of which the design of the floor and also the superstructure depends. With regard to the former, as previously stated (in Part I., on Weirs), the river coefficient adopted for the Nile is 125. That means that for safe frictional stability the effective weight of the superstructure and that of the floor must be 125 times that of the maximum statical pressure, both taken per foot run on the line of weir.

The area of the superstructure in one length of 7 metres is estimated at 280 square metres, with allowance for reduction of submerged portions.



(8b)

(8a)

FIGS. 8, 8a, 8b.—Assiût Regulator, Nile River.

The value of *A* per foot run will then be $\frac{280}{7} = 40$ cubic metres. Assuming the specific gravity at 2, and that of the cement concrete foundations at 2¼, the following is the analysis of frictional stability, *C* being 125:—

| | *A* | | *p* | | | |
|---|---|---|---|---|---|---|
| Rear floor . . . | 42 | × | 1 | = | 42 | Head, 2·55 feet. |
| Floor . . . . | 80 | × | 1¼ | = | 90 | |
| Superstructure . . | 40 | × | 2 | = | 80 | |
| Blocks . . . . | 14 | × | 1 | = | 14 | |
| Pitching . . . | 20 | × | ·8 | = | 16 | |
| Sheet piling . . . | 50 | × | ·4 | = | 20 | |

Total . . 262 ÷ 125 = 2·1
Balance, ·45 metre.

This discrepancy is due to the floor area having been calculated with the lower coefficient of 50, as adopted for the old Barrage apron, the weight of the head work being at the same time neglected as an asset. The system of using a higher coefficient and including the effective weight of the superstructure is clearly the more correct procedure, as has already been explained in Part I., Chap. VI. The weight of the superstructure is also underestimated. The ratio of $l : H$ (par. 21, Chap. VI.) is 30 : 1.

**(18)** With regard to the superstructure, the resultant line of maximum pressure should pass through the outer boundary of the middle third of the base of the piers. This test is best arrived at by graphical process and will be worked out for the Ibramiya Regulator, and so need not be repeated here. This determines the requisite base length of the piers.

The spans are 5 metres, *i.e.*, 16 feet, and the piers are 2 metres thick, a too high proportion of ·4*S*; every third pier is an abutment pier. The general arrangement closely resembles that of the Rupar Undersluice Head, but is even more simple. The working platform is carried on arches springing at high flood level. These arches are divided at the grooves, the open space being wider than usual. A travelling winch straddles this opening. The draw gates are of steel, fitted with anti-friction rollers, stanched by loose stanching rods as employed in Stoney's patent gates. The battered back to the piers is a decided improvement on the stepped back so common in old works in Upper India, although the rear face of the piers should be rounded, not square. For plans of gates, grooves, and travelling winch, *vide* Min. Pro. Inst. C.E., Vol. CLVIII.

The foundations were originally designed as rows of sunk blocks under each pier connected by a concrete floor. Owing to the successful introduction of cast iron tongued and grooved piling, the use of blocks was abandoned as being slow and expensive, and rows of sheet piling each side of the floor 25 feet deep were substituted, the depth of the floor being increased to a uniform thickness of 10 feet. This, as we have seen, is not in excess of the requirements of frictional stability, and the great depth is necessary to distribute the weight of the piers over the subsoil. Within certain limits it is immaterial whether the masonry area required for frictional stability be deep and narrow or shallow and wide, as long as the sectional areas are the same. So that we are enabled to give the full thickness of floor required underneath the superstructure, it all counting as part of the area of the floor proper. According to the rule adopted for weir aprons, the depth of the floor should be $\sqrt{H_1} + 1\frac{1}{2}$. $H_1$ in this case is 2·55 metres or 8·1 feet; the thickness would then be $\sqrt{8} + 1\frac{1}{2} = 4{\cdot}3$ feet. The additional thickness is required in order to reduce the unit pressure of the pier bases to a limiting pressure of under 1 ton per square foot, as has been fully explained in paras. 4, 5 to 7 and 10, Chap. IV.

(19) The detached masonry blocks shown in Figs. 1 and 1a are a comparatively new form of construction originally employed in the Punjab. These blocks are made of rubble masonry in cement mortar, and are laid in position when set, on the wet sand or in water by a heavy travelling crane. When *in situ* the interstices are filled up with cement concrete; it thus forms a solid mass of heavy masonry. This arrangement is admirably suited for a continuation of the masonry floor of a regulator. The advantage of the system lies in the fact that the blocks can be deposited in water, the interstices likewise being filled up by depositing cement concrete in skips, or else the work can be partially pumped dry to facilitate this operation.

(20) The plans of the Ibramiya Canal Head are given in Figs. 9.

This work, although a canal Head, is but a partial Regulator, and is consequently in a similar case to a river Regulator or an Undersluice Head.

The general arrangement of this work is identical with that of the Assiût Barrage.

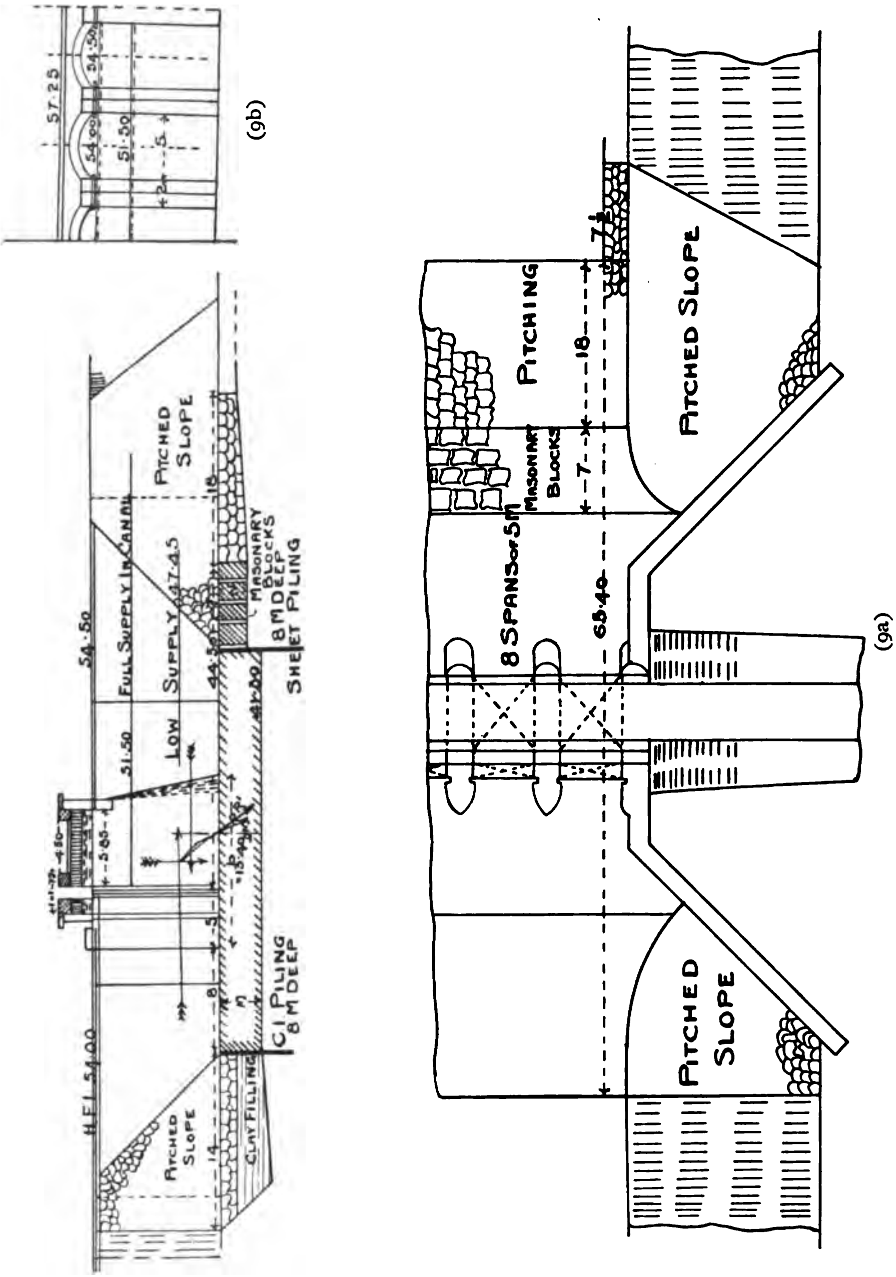
The Ibramiya Canal Head is, however, subject to a greater head of water than the latter work.

The maximum head of water to which the floor is subjected is the difference between low Nile level and the canal bed, *i.e.* 47·75 − 44·50 = 3·25 metres; so that this floor is under greater statical pressure than the Nile regulator. The superstructure is also subjected to a much greater pressure than the Nile regulator, owing to the great depth of water on both sides of the gates, which pressure is a maximum in Nile flood with full supply in the canal.

The transverse sections of the piers in either of the regulators are much the same, the base being 13·50 metres in each case; the top width is somewhat less in the Ibramiya work. The spans and thickness of the piers are the same in both cases, but the Ibramiya Canal Head has no abutment piers. The masonry floor is made wider in the canal head, being 31·40 metres against 26·50, a difference of 5 metres. This is owing to the greater head to which it is subjected.

The value of $A_p$ for the superstructure and floor of this work is estimated at 285, this divided by the coefficient 125, the quotient is 2·28, and the head being 3·25 leaves nearly 1 metre pressure unbalanced.

The reason for this deficiency is that according to the method adopted by the designers in estimating frictional stability of these two structures the effective weight of the superstructure was altogether ignored, that of the submerged floor alone having been considered, but with a reduced value for



FIGS. 9, 9a, 9b.—Ibramiya Canal Head Regulator, Assiût.

the river coefficient of 50. The writer has no hesitation whatever in stating that this method, not being founded on a sound basis of fact, must be wrong. As noted in Chap. VI., the design of the Damietta Weir is proof that in that recent case the old coefficient of 50 was not deemed reliable, and was very much exceeded in practice. The effective weight of the superstructure has probably also been underestimated.

As previously stated with regard to the sister work, the Nile Regulator, the proportional thickness of the piers to the span is undoubtedly excessive. This matter has already been dealt with in detail in Chap. IV. with reference to this very work (pars. 4 to 10).

### *Length of Floor beyond Head Work.*

**(21)** This, as we have already seen, is dependent more on dynamical than on statical considerations. Reference to the Table of Statistics with regard to various works shows a considerable difference in the width of masonry floors and also, in a less degree, in the thickness.

Among all the works the lengths of the masonry floors of the Sangam and Okhla undersluices are the least in proportion to their several heights of weirs. In the Okhla work the masonry floor is too short for safety, as a serious accident proved without doubt. Excluding these two, the general average width is $10H$, *i.e.*, ten times the height of weir crest above the floor. The extension of the talus should be from 10 to $12H$ further.

**(22)** With regard to thickness, the formula adopted for the floors or aprons of weirs will suit in this case, viz., $\sqrt{H_1} + 1\frac{1}{2}$, $H_1$ being the maximum statical head, generally greater than $H$ by 3 feet.

Where the head work is superimposed, the floor will necessarily have to be thicker in order to distribute the load better. This floor in moderate spans can be of one thickness throughout, as in the Assiût Regulator, or else, if the spans are larger, the piers can be separately supported on undersunk blocks or concrete enclosed with sheet piling.

The extra thickness required will be such that the pressure on the soil due to the superstructure will not exceed a desirable limit, as has already been explained in reference to the foundations of the Ibramiya Canal Head in Chap. IV.

A design of an Undersluice is given in Chap. VIII., Fig. 10, pars. 18 to 20.

**(23)** The previous examples are all of works founded on sand. At the heads of rivers, before the stream has left the hilly rocky country near its source and debouches into the plain, the river bed is generally composed of rock or boulders, the latter being more or less mixed with sand or shingle. It will be as well to give an example of a head work thus circumstanced.

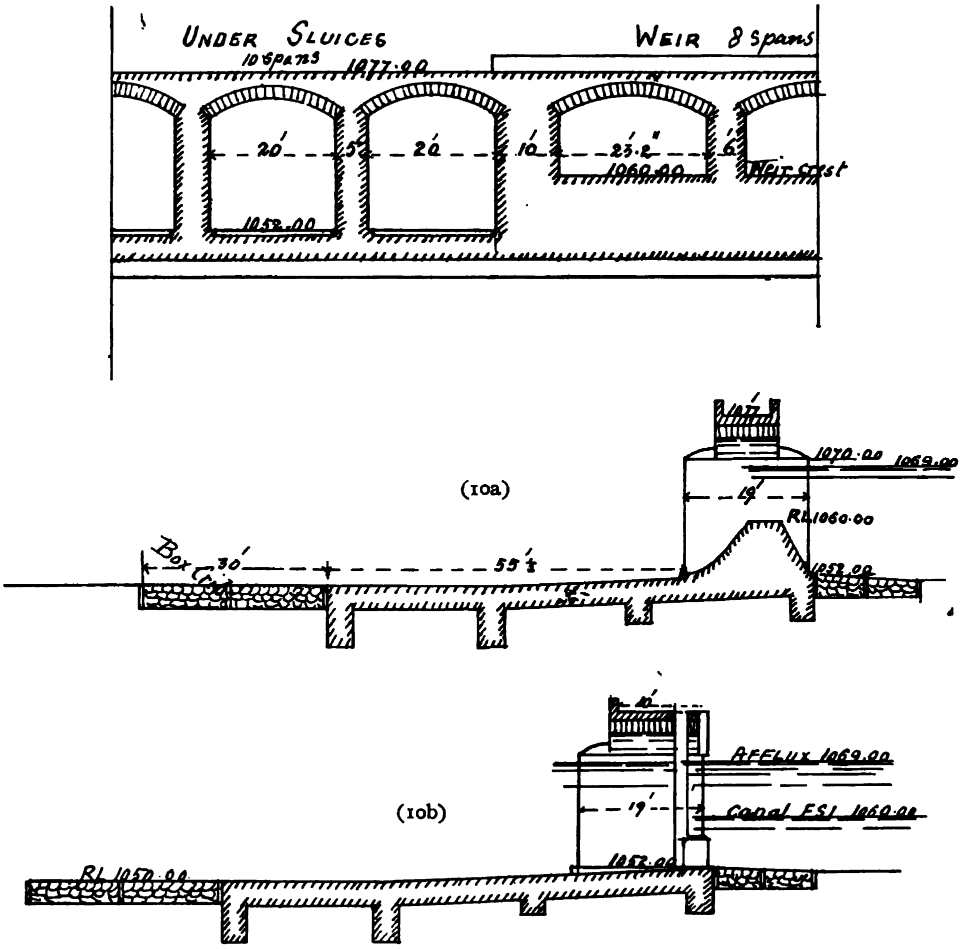
Figs. 10 are reproductions of the record plans of the Western Jumna Canal weir and Undersluices.

Of these Fig. 10 is a longitudinal section through part of the work. From this it will be seen that the high level bridge over the Undersluices is continued right along across the river as a road bridge, the weir being built inside the spans.

This cross communication is a necessary work, as canals take off on either flank of the river. The spans of the weir portion are 23 feet wide with 6 feet wide piers, whereas in the Undersluices they are reduced to 20 feet spans with 5 feet piers.

The Undersluices are partial regulators, and closure is effected by double iron draw gates without rollers, which bear against the sides of recesses formed in the piers.

The superstructure is provided with supplementary arches which are only 18 inches wide outside of the recess or grooves of the piers. The object of these external arches is the usual one of affording a narrow platform up stream of the grooves, which facilitates the manipulation of the draw gates. The afflux level is 17 feet above the floor of the Undersluices, while Full Supply Level in the canal corresponds to the crest of the weir, and is 8 feet above same level. The actual bare statical head on the weir and Undersluice floor is 8 feet. If one-third be added to allow for sudden freshets, high velocity of approach and other contingencies (*vide* para. 9), the



FIGS. 10, 10a, 10b.—Western Jumna Canal, Weir Undersluices.

maximum head can be assumed to be 10½ feet nearly, and will apply to the superstructure as well as to the floor.

(**24**) The sections of the floor of the Undersluices and of the weir proper are identical in length as well as thickness. In the previous examples the Undersluice floor is made much longer than that of the weir, and necessarily so, owing to hydrodynamical considerations. In this case, however, the boulder bed is capable of infinitely greater resistance to erosion than one of pure sand, consequently the talus can be curtailed; at the same time, owing to the great abundance of building material available at the actual site, the cost of a boulder masonry floor will be comparatively so inexpensive that the masonry apron can be carried wider than would otherwise be advisable. In this case the weir apron is about twice as long as it need be according to the rule already given of $3H_1$, whereas in the Undersluice the length from the grooves is 70 feet, which is less by one-third than what would be the correct length in case of sand.

(**25**) A work built on boulder formation cannot be considered entirely free from hydrostatic pressure below the floor. The coefficient of frictional stability must, however, stand at a considerably lower figure, than has been adopted for coarse sand, and we should be quite justified in reducing it tentatively from 75 to 60 ; probably a value of 50 would be nearer the mark.

The value of $A\rho$ in Fig. 10b is as follows (rear pitching assumed stanch) :—

| | | |
|---|---|---|
| Rear pitching . . | $50 \times 1 = 50$ | Head, 11 feet. |
| Superstructure . . | $103 \times 2\frac{1}{4} = 233$ | |
| Floor to pier toe . | $87 \times 1\frac{1}{4} = 107$ | |
| Total . . | $390 \div 60 = 6{\cdot}5$ feet. | |
| | | Balance, 4·5 feet. |

This is met by a floor thickness of 3½ feet, a specific gravity 1½, or over 5 feet of water, downward pressure.

The coefficient $C$ or $\frac{A\rho}{H}$ for the whole section works out to $\frac{760}{11} = 70$ nearly.

As regards thickness of floor, it is a little over $\sqrt{H}$, which would be 3·3 nearly. As the boulder foundation is really better than clay, the same rule of $\sqrt{H_1} + 1$ for thickness would answer, if taken as the maximum thickness below the bridge, tapering off to the recognised minimum of 3 feet at the end of the masonry floor, being thus of the trapezoidal section previously adopted.

Further, the thickness of floor given under the bridge is insufficient to properly distribute the load of the superstructure, and its increase is desirable. If the design were remodelled on modern lines it would resemble the Rupar Undersluice (Fig. 5), the only alteration necessary being the widening of the external separate arch from 18 inches to 3 feet and also of the slit opening to accommodate double or treble roller draw gates running in grooves or worked by a travelling winch.

This will lengthen the piers of the bridge by about 3 feet. The disposition of the canal Head with reference to the Undersluice is excellent and is illustrated in Figs. 16 and 17, Chap. VIII.

(26) In the United States a large number of canals have been constructed in recent years, the head works of which display considerable ingenuity in general design; some are diversion weirs, as dealt with in the last chapter, of low height, but the majority are constructed over the rocky beds of mountain torrents of considerable height, intended to form storage reservoirs similar in lines to the Betwa, Periyar, and Assuan works.

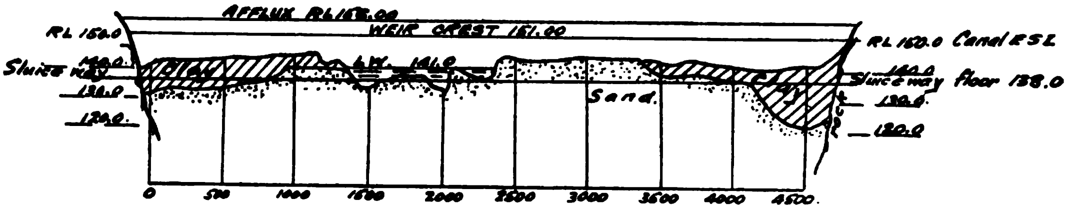


FIG. 11.—Section of Colorado River.

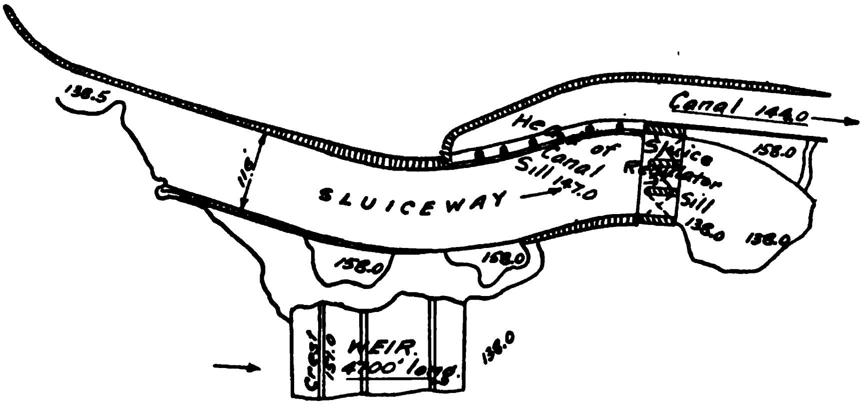


FIG. 11a.—Laguna Weir and Left Head Works.

One example of low weir across a wide sandy river has very similar conditions to Indian works. This is the Yuma Irrigation project about to be undertaken, a description of which is given in "Irrigation Engineering," an excellent American work, from which the plans have been obtained.

Fig. 11 is a longitudinal section of the Colorado River. From this it will be seen that the general level of the river bed is, roughly, about R.L. 143·0, the deepest part of the channel being about R.L. 131·00. The weir crest has been fixed at 10 feet above L.W.L., *i.e.*, at 151·00. The design of the weir is a close copy of the Madras anicut type, the fore slope being one in twelve with three concrete breast walls in the body. The level of the horizontal talus where the slope ends is R.L. 138·00, *i.e.*, 3 feet below L.W. The sluiceway floors on each side of the weir are at the same level, *i.e.*, 13 feet below crest. This level and that of the talus are deemed to be unnecessarily depressed.

The arrangement of the rest of the head works, viz., the Undersluices and canal Head, is so remarkable as to be deserving of considerable attention. The weir ends at both flanks on the existing rocky banks of the river. Beyond these and separated from the weir, sluice way channels are cut in the solid rock on either flank. These being quite independent of the weir, it is evidently a convenience to fix the position of the Undersluice head *below* the weir so that the junction of the scouring channel with the river bed will take place well clear of the latter work.

Another peculiar arrangement is the alignment of the two canals, which, instead of being, as is almost invariably the case in Indian canals, parallel to the axis of the weir, until the river banks are well cleared, are aligned parallel to the river and at right angles to the weir axis. This arrangement is necessitated by the level of the rocky ground in the vicinity of the head works. The canal Heads, therefore, which are situated just above the Undersluices in the side of the sluiceway channel, discharge through the right flank of the canal, *i.e.*, at right angles to its direction.

The sluiceway is kept at a very low relative level, viz., R.L. 138, in order to form a silt settling basin, to be periodically scoured out by raising the regulator gates. The sill level of the canal Head is at R.L. 147·00; thus a depth of no less than 9 feet of sand can accumulate in front of the canal Head, and even more, if double draw gates were adopted.

The afflux level above the weir is at R.L. 156·00, and, as, we have seen, the Undersluice sill is at 138, consequently 18 feet of water can be held up and released at once. These immense scouring sluices are evidently intended to do more than to keep the canal head clear of deposit, viz., to prevent sand accumulation at the rear of the weir and thus provide a reservoir of water to be drawn on.

**(27)** In a wide river like this, however, no scouring sluices, however powerful, will have any appreciable effect in keeping down the silt deposit in rear of the weir below crest level. The silting up of the whole river channel to the weir crest level, excepting a narrow strip at each flank, is *inevitable.*

Such undoubtedly being the case, the object of the Undersluices should clearly be limited to that of keeping a clear channel in front of the canal heads. This can be accomplished without incurring the great expense of excavating such deep sluiceways. In short, it is considered that the clearance of silt, so important an object in the design of Undersluices and canal Heads, has been overdone in the case under review. Very considerable economy without any sacrifice of efficiency could, it is believed, have been obtained by the following suggested changes in the design:—

The crest of the weir to be lowered 3 feet from R.L. 151, as it stands at present, to R.L. 148, and collapsible shutters of the usual type 3 feet deep adopted. The canal Head sill is at present at R.L. 147; this to be lowered 1 foot to R.L. 146·0, leaving a 2 feet instead of a 3 feet drop into the canal bed. The sluiceway floor, as well as the surface level of talus of the weir, to be at R.L. 141, *i.e.*, raised 3 feet higher. This will allow 5 feet to accumulate below the sill of the canal head and 7 feet if the draw gates are down. The Undersluice Head gates should be capable of closing up to R.L. 151·0, *i.e.*, a depth of 10 feet.

(**28**) With regard to the section of the weir, the author is strongly of opinion that this revival of the obsolete Madras anicut type is much to be deprecated. The river sand is probably of excellent quality, and the coefficient will certainly not exceed 75, if so much. Type D should be followed, the horizontal apron floor being placed at R.L. 142·00, the masonry weir being 6 feet high only. The surface of the apron could be pitched dry with the slabs it is now proposed to use for the long fore slope. A row of sheet piling 12 feet deep to go right across up stream of the weir wall. This should be deepened to 20 feet or more, where the river channel is crossed, and a second row used down stream, the river being diverted and the depression filled up to R.L. 142·00 with the clay and sand obtained from the excavation of the rest of the weir. The action on the weir thus altered will be very moderate, as only 5 feet of water is calculated to pass over at full flood, so that with the adoption of type D a comparatively short width of protective talus pitching will be required, the floor proper being 40 feet wide, of trapezoidal section. The existing design, modelled on type A, has a width of 244 feet, which is much in excess of requirements, even with this type, unless the river sand coefficient stands at 100 or over.

The diversion weir proper and the storage weir or dam are naturally not very distinct, particularly in American works, where, however, most are of the latter type, combining storage with diversion of a perennial or intermittent stream, as the case may be. Some of these are treated of in Chap. XIII.

(**29**) The term Undersluice, familiar in Northern India, is in reality a misnomer, as in modern weir designs, undersluices, literally as such, do not exist. Scouring sluice Heads would be a more reasonable term.

In the United States, where the nomenclature differs somewhat, the term Undersluice applies only to Weir or Dam *body* sluices—as those in the Assuan Dam.

## STATISTICS OF UNDERSLUICES.

| *H* = Height of Weir plus Shutters. | *T.* | *L.* | *LP.* | *RL.* | *DP.* | Ventage. | Flood Discharge of River cubic feet per sec. | Ventage of Regulating Canal Head. | Regulator Sill above U.S. Floor. | Name of Work. | River. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 13¾ | 2 ? | 127 | 300 | 14 | 2 ? | 15 of 5½ ft. | 1,210,000 | 15 of 6 ft. | 4 | Dauleshwiram | Godaveri | Below the 2 ft. of masonry is probably 4 ft. of pitching. |
| 13 | 6 | 90 | 94 | 17 | 6 | 14 of 6 ft. | 597,600 | 14 of 6 ft. | ? | Sangam | Pennér | |
| 11 | 6½ | 122 | 155 | 25 | 6½ | 20 of 20½ ft. | 830,000 | ? | *nil* | Dehri | Són | |
| 13¼ | 4½ | 114 | 160 | 27 | 12 | 38 of 5 ft. | 900,000 | ? | 1 | Jobra | Mahanadi | |
| 10½ | 4 | 90 | 67 | 10 | ? | ? | ? | ? | ? | Brahmini | Mahanadi | 1½ ft. of the floor is of rubble filling. |
| 13 | 6 | 72 | ? | ? | ? | 16 of 6 ft. | 150,000 | 16 of 6 ft. | 4 | Okhla | Jumna | |
| 13 | 5 | 155 | 100 | *nil* | 9 | 42 of 7¼ ft. | 350,000 | 25 of 7 ft. | 3 | Narora | Ganges | |
| 10 | 4 | 96 | 100 | ? | ? | ? | ? | ? | ? | Barra | Mahanadi system | |
| 12 | 4 | 100 | 150 | 15 | 5 | 12 of 20 ft. | ? | 39 of 5 ft. | 2 ft. | Rupar | Sutlej | Boulder bed. |
| 8 | 3½ | 60 | 30 | 15 | 3½ | 10 of 20 ft. | ? | 35 of 6 ft. | 4 ft. | Faizabad | Jumna | Do. do. |

*H* = Maximum statical head of water.
*T* = Thickness of floor.
*L* = Width of floor beyond head work.

*LP* = Width of pitching beyond.
*RL* = Extension of floor in rear of sluice head.
*DP* = Depth of foundation of piers of headwork.

# CHAPTER VIII.

## CANAL HEAD WORKS.

### PART III.—CANAL HEAD REGULATORS.

(1) THE function of canal Head Regulators is to adjust the admission of water into the canal from the river of supply, according to the requirements of the canal.

In almost all cases this regulation is entire, not partial; that is to say, on the one hand, the full supply level in the canal cannot be allowed to be exceeded, and on the other hand, the head work should be capable of completely shutting off all supply, even during the highest floods in the river. From the above it will be evident that the pressure of the statical head of water to be provided against is much higher in this class of work than in Undersluices, while the dynamical forces which are such potent factors in the latter are practically absent in Head Regulators.

The admissible mean velocity of water in a canal rarely exceeds 3 cubic feet per second, and the waterway of the head is generally made ample in width in order not to cause increased velocity of entry, so that the only occasion when the velocity of entry exceeds the normal is when it passes over the lower of the pair of draw gates, as in a submerged fall, or over a raised sill.

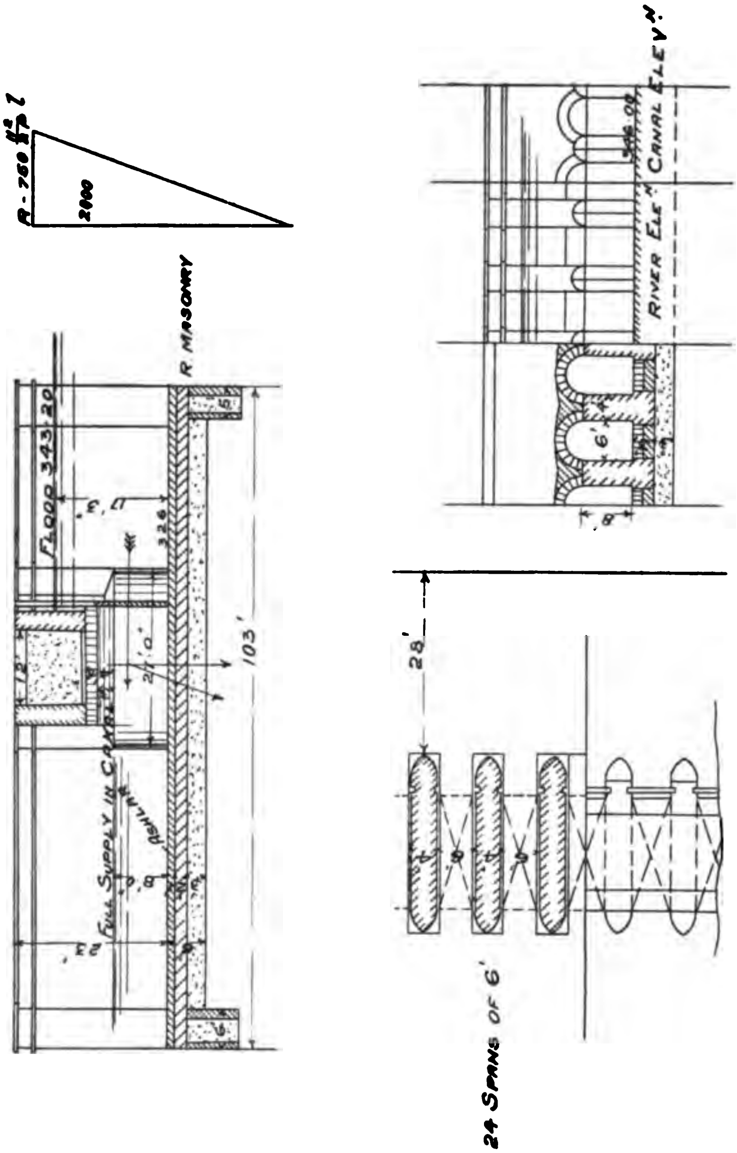
(2) The following principles regulate the design of canal Heads:—

First: The width of the openings should be as large as is consistent with easy manipulation of the draw gates, which are liable to be subjected to much greater water pressure than can be the case with limited regulation.

Secondly: The height of the vents, *i.e.*, the spring of the arches, should be at canal Full Supply Level (F.S.L.). Above this level, a breast wall must be carried up to above Maximum Flood Level (M.F.L.) in the river. The segments of the arches must likewise be closed either by panel walls resting on the top sill of the iron gate frame, or else this closure is effected by the upper draw gate covering the segment, by bearing against a horizontal bar projecting from the masonry. The gates consequently must be watertight, not only laterally, but also at the upper sill.

(3) We will now proceed to examine a few existing examples of these works. The first will be the Dehri Head Regulator on the Sôn Canal (Bengal) (Fig. 1).

The maximum head which acts on the floor of this work is 17½ feet. The area of the masonry floor (which is 6 feet deep), excluding the curtain wells, measures 103 × 6 = 618 square feet per foot run. Taking the value of ($\rho$ − 1) as unity, the resistance offered by the floor is 618 square feet; add 137 for curtain walls plus lateral influence, and $\frac{2,000}{9} \times 2\frac{1}{4} = 500$ for the



(1b)

(1a)

FIGS. 1, 1a, 1b.—Dehri Canal Head Regulator.

superstructure, we obtain a total value of $A\rho$ per foot run of 1,255. This should equal $CH$ or $75 \times 17\frac{1}{4} = 1,294$, a close correspondence. (For this class of river $C$ is 75.)

With reference to the upward pressure exercised on the floor just beyond the head work: In the floor, fore curtain and superstructure, up to this point, $A\rho$ has a value of 800. Dividing by 75, the loss of head will be

10·7 feet, leaving 17½ - 10·7 = 6·5 feet, balance of head. To counteract this the effective depth of the floor is equivalent to 6 feet of water, leaving ·5 feet unbalanced. There must, however, be pitching for a great width in front of the Head not shown in the drawing, which will be a further asset not included in this calculation.

(4) The projection of the floor to the rear of the work is considerable, as it fills up the large bay formed by the curved wings which connect it with the undersluices. The noses of the cut-waters are thus recessed 30 feet behind the line of the abutment of the undersluice. This arrangement can best be understood by reference to the block plan (Fig. 6c, Chap. VII.). The bay thus formed is a veritable silt trap, and, as already observed elsewhere, must largely assist in the deposit of silt in the canal bed, the removal of which costs an immense annual sum; otherwise in this class of head work the rear floor projection is an advantage. A long, heavy floor is required for purposes of frictional stability, and as the whole of this length is not required in the canal side of the work for dynamical considerations, it is of advantage to place as much as possible on the rear side, in order, with regard to upward pressure, to reduce the effective head at the critical point on the floor just below the piers. This could be effected by continuing the masonry floor (or deep pitching) across the Undersluice approach channel. This floor will be at a lower level than that of the Head, viz., level with the Undersluice floor. Further, a row of deep steel sheet piling at the junction of the two levels will be a necessary adjunct. This sheet piling in the rear of the work is as indispensable as that at the downstream end of an Undersluice floor.

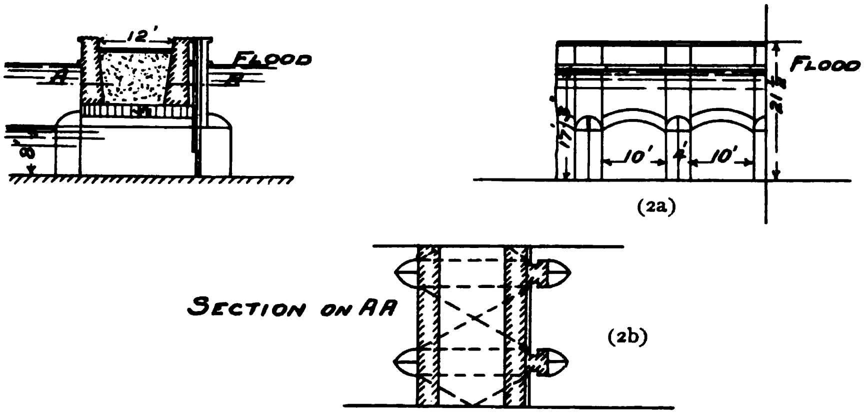
(5) The superstructure of the Dehri Regulator is of what may be termed the U section, *i.e.*, the arches are depressed to canal Full Supply Level, which is 8½ feet above the floor, and two longitudinal breastwalls are built above the ends of the vaulting, enclosing a space which is filled up solid with clay or stone.

The grooves for double wooden gates are outside the rear breast wall, and square pilasters are carried up above the cut-waters, in the sides of which the grooves are recessed. This arrangement is simple and effective. The fault of the design lies in the smallness of the openings. They could very well be made 10 feet or 12 feet wide instead of 6 feet, and the gates be fitted with anti-friction rollers.

The piers are also too thick, being $\cdot 7S$, whereas a proportion of $\cdot 4S$ would be ample. The road level seems also unnecessarily high above M.F.L.

(6) These points have been corrected in the revised design of Fig. 2, as well as the profile of the arches and the footing of the piers, which are faulty. In Fig. 1 the arrangements for lifting the gate are not shown. A travelling winch could work on the roadway with arms projecting over the side, to which pulleys are attached, and so hoist the gates. This arrangement is not uncommon. It, however, has the disadvantage of blocking the roadway, which is required generally to pass traffic.

This suggested modification of the Dehri Head Regulator is based on the supposition that the sill level of the Head Sluice in respect to that of the Undersluices is as it should be. As has already been pointed out, this is by no means the case. The design of the existing head works is such, that a radical remodelling of the whole would be most desirable. As matters are at present, sand is washed into the canal to such an extent that a heavy permanent establishment with steam dredgers has to be employed to keep the channel at all clear of obstruction. The most of this annual expenditure, which saps all the profit of the undertaking, could be saved by a drastic reconstruction of the head works on the following basis: Undersluices to be rebuilt with 20-feet spans on the model of Rupar, with high-level platform and roller draw gates. Head sluices to be provided with a dwarf head weir wall 4 feet high as at Jamrao head; spans increased to 12 feet or 15 feet, with arch spring line somewhat raised. Weir crest to be raised in proportion; probably 1 foot would be sufficient, with deeper crest shutters.

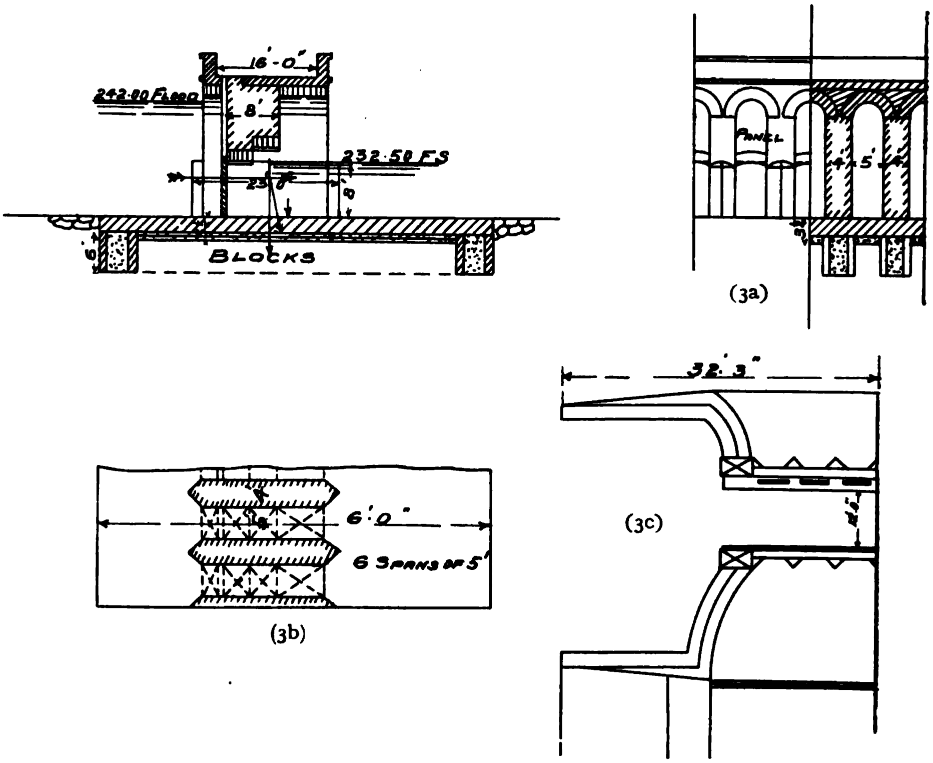


FIGS. 2, 2a, 2b.—Alternative Design.

(7) The Narora Head Work (Fig. 6, *post*) is of similar design, except that the roadway is carried on jack arches instead of being filled up solid. In this design however, external arches are built outside the grooves on continuations of the cut-waters, as is common in Undersluices. These arches carry an outer rail for the traveller, the inner resting on the parapet. The foundations of this work appear insufficiently long for a sand soil. The head is 15 feet, *i.e.*, 2 feet less than in the last example, and length of the masonry floor is 52 feet only, *i.e.*, half that of the Dehri Regulator. It should really be longer, as the river coefficient of the Ganges is 100 against 75 of the Son. This discrepancy can, however, be accounted for, as the foundations of the Head Regulator at Narora rest probably on solid clay.

(8) Another example is given in Fig. 3, that of the Saran Canal Head, in which the flood level and canal supply levels are almost identical

with those obtaining at the Dehri Regulator. The superstructure, however, is not on the so-called U design with depressed arching, but has high piers and an arched platform, split for the grooves above M.F.L. In addition, a lower and narrower tier of arches springing at canal Full Supply Level, support an immensely thick breast wall, which is carried up to the upper archway and so blocks the waterway above the gates. The narrow spans of 5 feet have piers 4 feet thick, a proportion of ·8*S*, and are thus grotesquely massive. The general arrangement is not dissimilar from that of the Sangam Undersluices already described in Chap. VII.

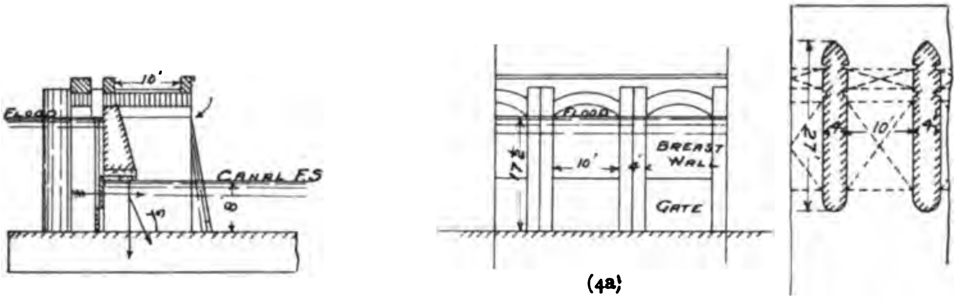


FIGS. 3, 3a, 3b, 3c.—Saran Canal Head.

The thickness of the breast wall is very excessive. If cut in half and placed on either flank, forming a U section, the lower tier arches being widened accordingly, it would still be wastefully thick; but if this arrangement were adopted, the main high-level arch, the piers above the level of the lower arch tier would be entirely saved.

(9) Figs. 4, 4a and 4b represent a revised design for the same work, but on identical lines.

The spans are doubled in width to 10 feet, and the breast wall is shown carried on rolled beams built round by concrete. This arrangement avoids the hollow segment of the intrados of the arches of the lower tier, and enables the upper gate to bear against the flanges of the outer beam, thus forming a simple watertight joint. In Fig. 4, outer spare grooves are provided into which baulks may be thrown to close the bay independently of the gates, in case of accident or other necessity. These are provided in the Assiût Regulators, but their adoption in a canal Head work involves lengthening the cut-water up stream, which is objectionable, as if the piers project much beyond the gate grooves, the scouring action of the Undersluices is hindered and a slit trap is formed. In addition to which, the portion of the piers to the rear of the grooves are subject to flotation, and consequently the material is half as effective from a statical point of view as if it were disposed in front of the gates, and would obviously be better transposed below.



(4a)

(4b)

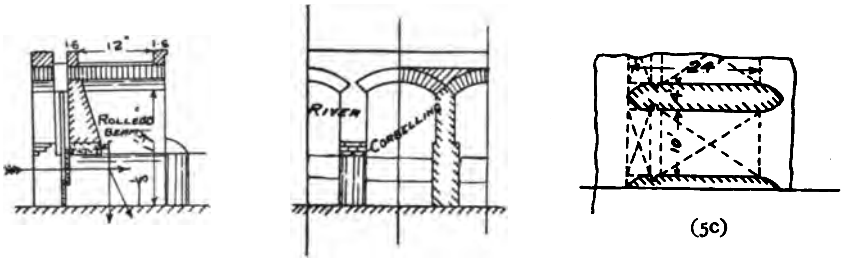
FIGS. 4, 4a, 4b.

This shortening of the cut-waters has been effected in Fig. 5. There being no current above the top of the vents, a cut-water is not required above that level; the pier in this case is shown corbelled out square above canal Full Supply Level and continues of rectangular section to the summit, where it supports the narrow external arch. The material saved both by this means and by the abandonment of the spare grooves is utilised for the rear, by widening the roadway. The areas of the two sections Figs. 4 and 5 are almost identical.

**(10)** To afford comparison of relative cost, a statement of the quantities in all five sections, Figs. 1 to 5, is given below. These quantities are the superficial area per foot run :—

| Design. | Quantities of Masonry in one Span. | Divide by Length. | Quotient. | Reduced Number of Comparison. |
|---|---|---|---|---|
| Dehri, Fig. 1 | 2,161 cubic feet | 10 feet | 216 | 110 |
| Revised, „ 2 | 2,460 „ „ | 14 „ | 176 | 90 |
| Saran „ 3 | 2,682 „ „ | 9 „ | 300 | 150 |
| Design „ 4 | 2,772 „ „ | 14 „ | 198 | 100 |
| „ „ 5 | 2,750 „ „ | 14 „ | 196 | 100 |
| „ „ 5b | 2,600 „ „ | 14 „ | 185 | 95 |

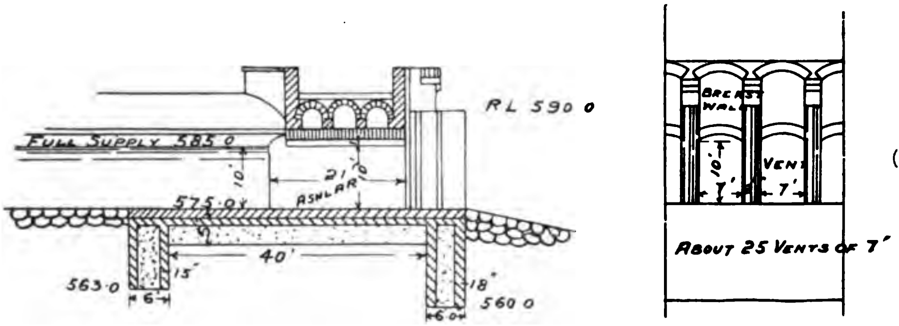
The conditions of flood level and canal supply are practically identical in all cases. Fig. 2, of U section, is the cheapest and Fig. 3 of the Saran Canal is naturally the dearest by far. If the reduced section of pier shown in 5b be adopted, the cost of the 2a type will approximate to that of the U-shaped section, so that is almost immaterial which form is adopted. The difference in cost of the original works (Figs. 1 and 3) and that of the revised designs (Figs. 2, 4 and 5), is really greater than what is exhibited on the Table, for the waterway could well be reduced with the larger vents employed, the coefficient of discharge being higher owing to the reduction in side contraction, and fewer piers are required.



(5a) (5b) (5c)

FIGS. 5, 5a, 5b, 5c.

The incidence of the resultant line of pressure has been drawn on Figs. 4, 4a and 5. In every case it falls well within the base. If a width of roadway not less than 12 feet be adopted, the requisite base length of the piers will generally be arrived at automatically.



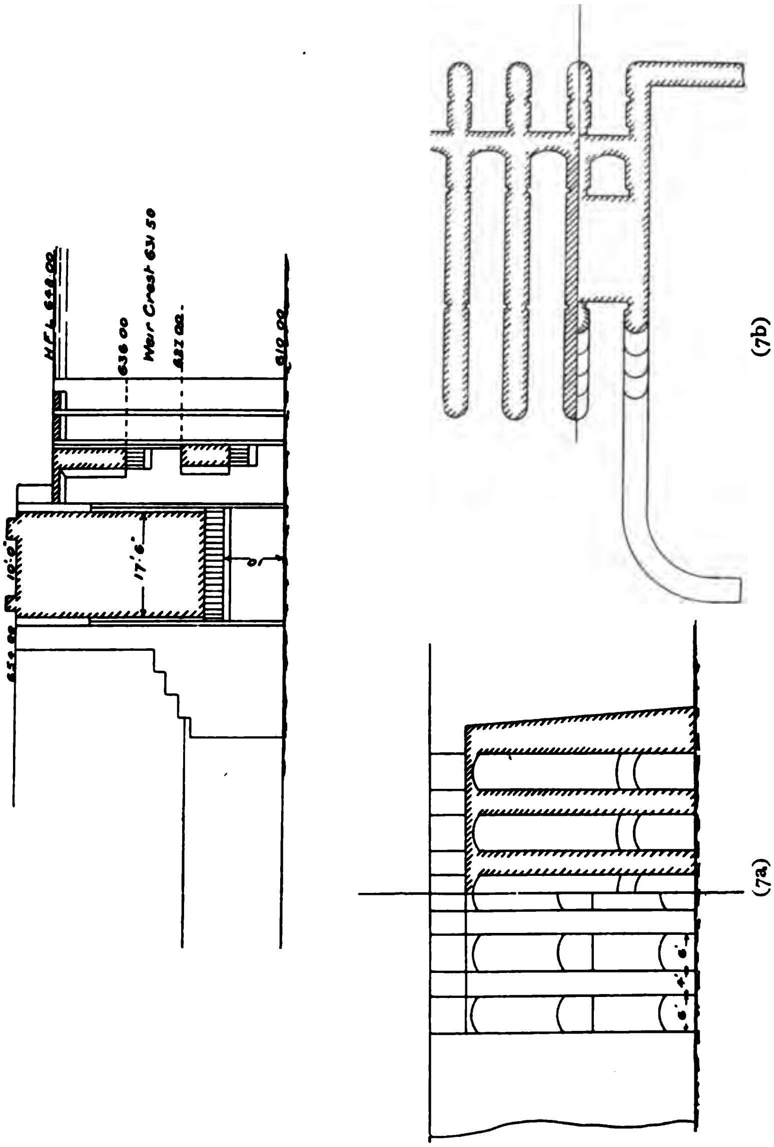
LONG^L SECTION RIVER ELEVATION

FIGS. 6, 6a.—Narora Head Regulator, L. G. Canal.

**(11)** The Narora Head Work (Fig. 6) was the first work supplied with gates fitted with rollers, or rather wheels, the axles of which work in bearings fixed to the gate. The stanching of these gates is effected by cutting the table of the vertical groove, on which the wheels run, to a certain batter at the lower end, so that when the gate reaches the sill its sides and top are in close contact with the iron frame.

The adoption of the usual round or flat stanching rods would be a simpler and equally effective arrangement. The gates are raised by a travelling winch, outside arches being provided for carrying one of the traveller rails, the other resting on the parapet of the bridge. These gates are double.

The design of this work is excellent, excepting the width of the openings, which, under so moderate a head of water, could well have been enlarged to 10 feet spans, with piers 3½ feet thick.



FIGS. 7, 7a, 7b.—Betwa Canal Head Regulator.

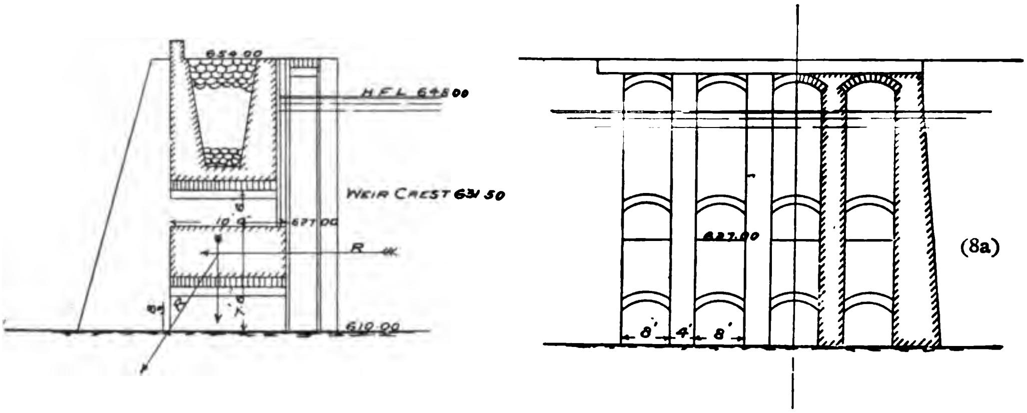
**(12)** An interesting example of a canal Head of exceptional design is given in Fig. 7 of the Betwa Canal Head.

This work takes out from a reservoir formed by a high weir across the

River Betwa in Upper India, and is subjected to a head of no less than 38 feet of water above the sluice floor. As anti-friction roller gates were hardly known at the time of its construction, special devices had to be employed to modify the pressure of water against the sluice gates so as to enable them to be manipulated under such severe conditions. This was effected by building a panel screen wall in front of the face of the Head work, which was perforated by two sets of vents at different levels, closed by draw gates, one in each bay of the regulator.

These panel walls are arched in plan, somewhat similar to those in the Assuan Dam, and are carried by the vertical arches over the vents. The upper portion is recessed somewhat in rear of the face of the lower to afford a ledge for the upper drop gate to rest on.

When the reservoir is sufficiently full, the lower outer vents are closed, and water for the canal is taken through the upper outer vents, which falls into the space between the outer screen wall and the face of the regulator.



(8a)

Figs. 8, 8a.—Alternative Design.

The arrangement is clearly illustrated in Figs. 7, 7a and 7b.

This device, though ingenious, is expensive, and with the universal adoption of anti-friction rollers to draw gates, or Stoney's apparatus as employed in the Assuan Dam sluices, which reduce the power required to lift a gate by $\frac{1}{80}$ to $\frac{1}{250}$, is not likely to be repeated.

(**13**) As, however, it is often more convenient, in order to avoid silt depositing in the canal, to tap the river water from a high level, there is no reason why, even under like conditions, the head work should not be pierced by a double set of sluice vents at different levels.

This arrangement would be just as effective as the last, and would certainly be much cheaper. It is exemplified in Figs. 8 and 8a. The immense solid mass in Fig. 7 is unnecessary for purposes of stability, and can be lightened with advantage by the upper sluice openings, and further by adopting the U section above them, the central space being filled with loose stone or rubbish.

In this design, when the upper vents are open and the lower closed, the water will fall directly into the canal, down stream of the work. There is no disadvantage in this, particularly at this site, where the canal is in rock cutting, a small extension of the floor and wings being all that can be necessary. It is suggested that this arrangement of double sluices, which, as far as is known, has hitherto not been tried, is worthy of adoption in all head works where the river in flood is very deep and where silt deposit gives trouble. The silt in suspension could be allowed to accumulate in front of the Head work to a considerable depth, and then by opening the Undersluice gates it could be swept out, leaving a clear channel ready for the time when the water level has fallen, and supply is drawn from the lower gates.

**(14)** In order to prove that the section of the Head work in Fig. 8, with spans enlarged to 8 feet as in Fig. 8a, is sufficiently wide and heavy to withstand the water pressure, the incidence of the resultant line *R*, on base of the profile (Fig. 8), is shown. It will be seen that it falls just within the middle third of the base of the piers.

As noted already, the spans have been increased from 6 feet to 8 feet, the pier thickness being left at 4 feet. The width of the main work is made the same as before—but buttresses are thrown out in rear as continuations of the piers, similar to the Assiût Regulators. The head on this work being now 44 feet, owing to the newly-adopted 6 feet high weir shutters, it has probably been also raised 6 feet. A design on same lines is Fig. 22.

With the adoption of roller gates, the lifting apparatus can consist of a travelling winch spanning the groove openings, the external arching being provided for that purpose. In the existing work, screw lifting gear is used, and that of a primitive pattern, the female screw, held in collars, having the motive power applied, while the male screw rod attached to the gate rises through it. These are worked with great difficulty. The absence of roller attachment necessitates the use of screws to force the gates down against the water pressure, whereas where rollers are provided the gates drop into place of their own weight and only require gear for raising.

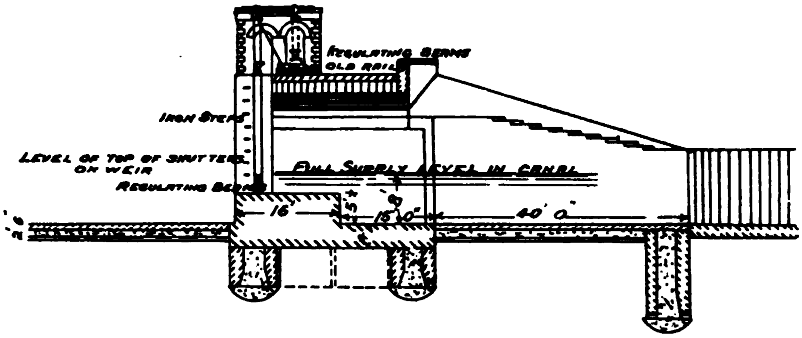
**(15)** It often happens that a canal must take off from the river at a low level, either in order to save a masonry fall in its bed or to economise the height of the weir.

In order to effect this and yet at the same time have the floor of the Head at a sufficient height above that of the Undersluice to allow of the alternate deposit and scour of silt, the sill of the canal Head is raised above its general floor level. On this dwarf weir the gates rest. This raised sill diminishes the waterway but affects the discharge of entry in a very moderate degree, as the increased velocity of entry due to the submerged fall nearly compensates for the diminished waterway. If the waterway of the vents were proportionately increased laterally, there would be no appreciable diminution of discharging capacity, and consequently the height of the weir and Undersluice gates need not be raised above what would be requisite if the raised sill did not exist.

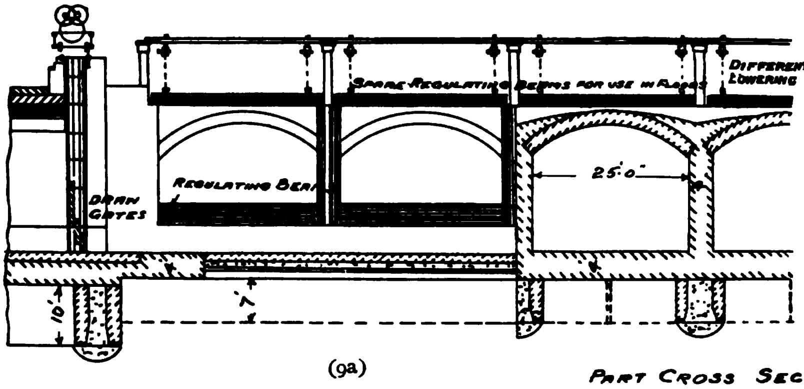
**(16)** An excellent example of this is given in Fig. 9 of the Jamrao Canal Head lately constructed on the Naza, an outlet channel of the Indus River.

In this design the canal bed is at the same level as the floor of the Undersluices. Across the Head a dwarf weir is built 5 feet high. The canal full supply is 8 feet deep and the crest of the weir shutters is 10 feet above floor, consequently 5 feet of water can pass over the raised sill crest, which will afford the supply required in the canal.

For the purpose of diminishing end contraction in the submerged film passing the dwarf weir, the openings have been made very wide, viz., 25 feet spans, of which there are six. The springing of the arches is at maximum flood level, *i.e.*, at 15 feet above canal bed, consequently the closure of the 10 feet space above the sill has to be effected by baulks or regulating beams lowered into grooves and raised by means of differential tackle. The Undersluices shown in section in Fig. 9b, consist of seven spans of 20 feet; total ventage, 140 feet against 150 feet of the canal head. The partial regulation of the Undersluice is up to a depth of 10 feet and is effected by three draw gates manipulated apparently by fixed winches at each pier head.


REGULATING BEAMS OLD RAILS
IRON STEPS
LEVEL OF TOP OF SHUTTERS ON WEIR
FULL SUPPLY LEVEL IN CANAL
REGULATING BEAMS
16'
5'
8'
15' 0"
40' 0"
2' 6"

LONG<sup></sup>



(9a) PART CROSS SECTION OF CANAL HEAD.

(9b) LONG^L SECTION THRO' UNDERSLUICES

FIGS. 9, 9a, 9b.—Jamrao Canal Head.

**(17)** The fault of the design of the canal Head lies in the slowness of regulation necessitated by the width and height of the openings. Complete closure has to be entirely effected by the regulating beams. It is believed that the design would be improved if the spans were reduced to 15 feet and correspondingly increased in number, with flat arches of a versed sine of 2 feet springing at 10 feet above floor. Two rolled beams to be built in at spring line in each span for the top gate to abut on and for the purpose of carrying panel walls, closing the open arch segments. This will leave 5 feet depth for regulation, which could easily be effected by single or double roller gates raised by a travelling winch.

The abutment spans of both Regulator and Undersluices should be tied by iron rods.

It is quite evident that a wide and high bridge opening is not suitable for a canal Head work though it is so for Undersluices. We have already seen in Chap. VII. that the design of the Sangam Undersluices would be better adapted for a canal Head than for the purpose for which it is intended, while on the other hand the Jamrao canal Head is more suitable for an Undersluice or a partial Regulator across a river than for a canal Head.

**(18)** Fig. 10, etc., is a design for a combined Undersluice and Head Regulator under assumed conditions, similar to those in the Dehri Head Regulator.

The flood level is 20½ feet and the canal bed is 3 feet above the undersluice floor. The Undersluice gates will hold up 12 feet of water. The full supply in the canal is 8 feet deep. The sill of the Regulator is raised 2 feet above the floor by a dwarf weir wall. This leaves a depth of 5 feet in the Undersluice approach channel for the deposit of silt.

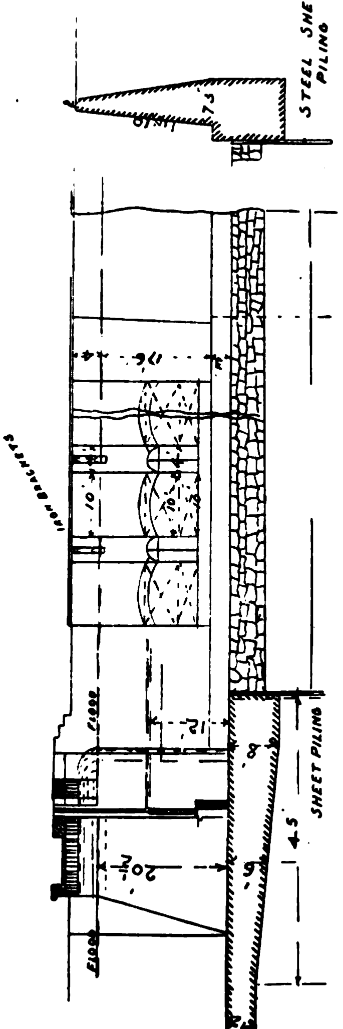
The Regulator (10b) is of the U type, with depressed arches, the vents being 10 feet wide. Double gates are provided, to be lifted by a traveller running on rails; one rail is on top of the rear breast wall, while the other is supported by beams resting on iron brackets bolted to the pilasters of the face.

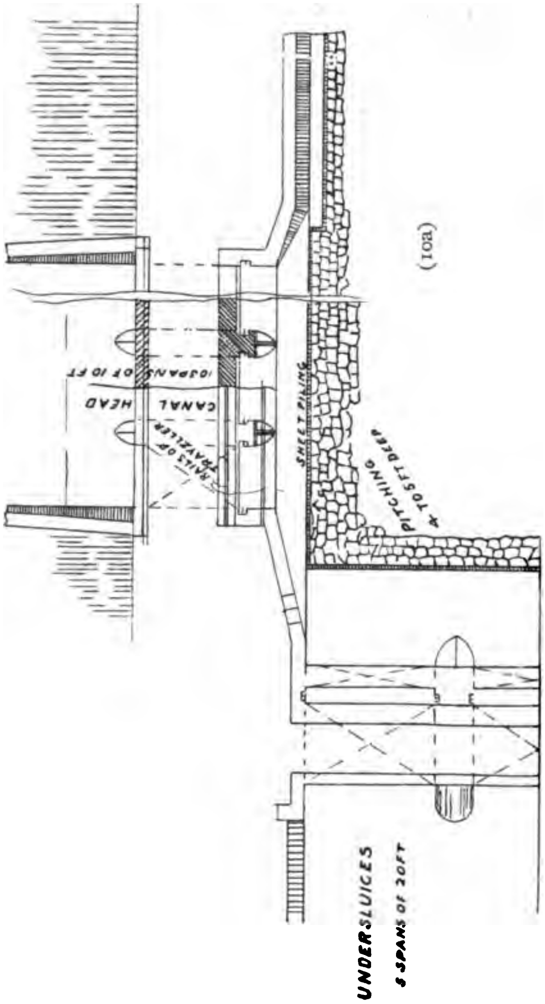
This arrangement obviates the lengthening of the piers, which would be necessary were an external arch provided.

The plan of the Undersluices is on the lines of the Assiût regulators and Rupar Undersluices.

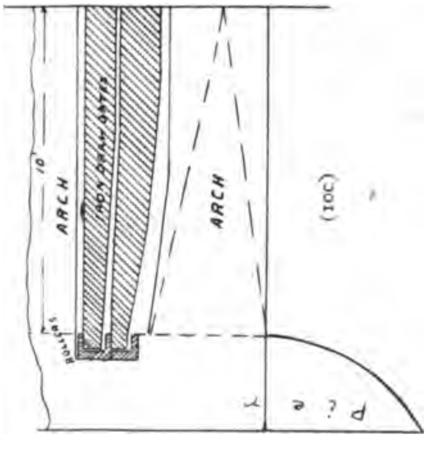
The canal Head cut-waters are recessed 5 feet behind the face line of the abutment of the Undersluice. The connection between the rear abutments of either work is effected by a straight vertically faced revetment wall. All the other wing walls are battered. In order to avoid a corner at the junction of the battered right flank wall with the right abutment (Fig. 10a), the splayed connecting wall starts with a vertical face, which is gradually inclined backwards till it meets the flank revetment, where its batter coincides with that of the latter, which is 1 in 10. This gradual change of face slope is an excellent device, as it obviates sharp shoulders, which are always an objectionable feature in irrigation works.

The down stream wings run out parallel to the centre line of the canal with sloping crests, and extend to the end of the masonry floor.





(10a)



(10c)

Figs. 10, 10a, 10c.—Design for Undersluice and Canal Head Works.

(19) The dimensions of the floor of this work are designed in accordance with the principles laid down in Chap. VI., viz., that the effective weight of the superstructure and floor must be sufficient to neutralise the head acting on the work, which is 17·5 feet.

First, with regard to the superstructure of masonry and filling. The area of one span of 14 feet is estimated at 4,000 square feet. This is supposed to be of brickwork of a specific gravity of 1·84.

The value of $A\rho$ for the whole structure, per foot run transversely, will be $\frac{4,000}{14} \times \rho = 285 \times \rho$ nearly $= 285 \times 1·84 = 524$.

Assuming the river coefficient at 75, the head neutralised due to the superstructure will be $\frac{524}{75} = 6·99$, say 7 feet, leaving a balance of 10·5 feet.

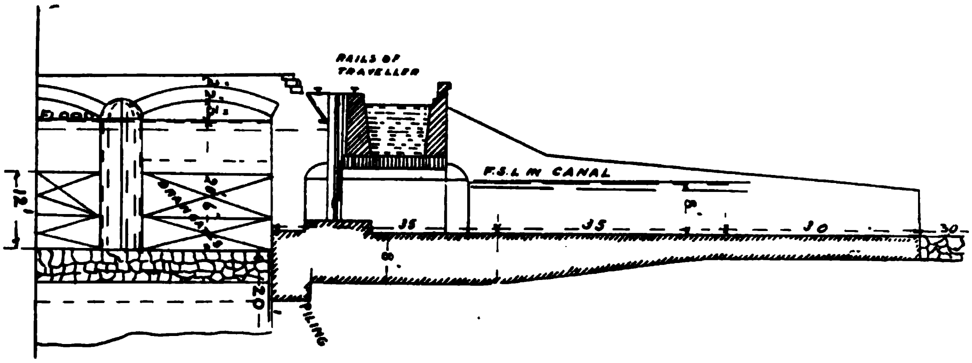


FIG. 10b.—Design for Undersluice and Canal Head Works.

Now the value of $A\rho$ of the floor up to the break in foundation is as follows:—

| | $A$ | $\rho$ | $A\rho$ |
|---|---|---|---|
| Sheet piling . . | 100 × | ·6 = | 60 |
| Floor . . . | 300 × | 1 = | 300 |
| Total . . | | | 360 ÷ 75 = 4·8. Balance, 5·7 feet. |

The downward pressure is that of 8 × 1 = 8 feet.

Floor continued:—

| | $A$ | $\rho$ | $A\rho$ |
|---|---|---|---|
| To end of inclined base . | 210 × | 1 = | 210 ÷ 75 = 2·80. Balance 2·9 feet. |
| To termination . . . | 120 × | 1 = | 120 ÷ 75 = 1·60. Balance 1·3 feet. |

The length of pitching 4 feet deep required will be obtained from the formula $x = \frac{CH}{d\rho} = \frac{75 \times 1·3}{4 \times ·8} = 30$ feet ($d$ is depth of pitching).

The heavy foundation given in this design is due to the low specific gravity adopted for the masonry of the work. If this were raised to $2\frac{1}{4}$, the foundation would be much less in area. If the weight of the superstructure were neglected and the floor alone included in the calculation, using a coefficient of 35, as was done with the Assiût Regulator, the result would be very much the same. The ratio $l : H$ is $\frac{190}{17 \cdot 2} = 1 : 12$ nearly.

The pressure on the foundation, owing to its great depth, will be only about half a ton per square foot.

A better arrangement of the position of the canal Head would probably be to shift this work bodily forward up to the line of the abutment of the Undersluices, and to have a wide paving in front, in the sluice-way approach. This would enable the floor of the Head work to be reduced, material taken from below being transposed in front. A dwarf partition wall is required on the weir side to define the approach channel.

(20) With regard to the Undersluices, the area of masonry resting on the base of each pier is estimated at 4,500 square feet, representing a load of $4{,}500 \times \frac{1}{36} \times 1 \cdot 84 = 230$ tons. The piers being $30 \times 6 = 180$ feet superficial area, the pressure at their base will be $\frac{230}{180} = 1 \cdot 3$ tons per square foot, and at the base of the floor will be $1 \cdot 3 \times \frac{6}{6+6} = 1 \cdot 3 \times \frac{1}{2} = \cdot 65$ tons per square foot only.

Taken transversely, the weight of the superstructure will be represented by $\frac{4{,}500}{26} \times \rho = \frac{8{,}280}{26} = 318$. This is the value of $A\rho$ per foot run.

The reduction of head, which is 12 feet, will be as follows:—

| | $A\rho$ |
|---|---|
| Superstructure . . . | 318 |
| Floor to break . . . | 270 |
| Sheet piling . . . . | 60 |
| Total . | 648 $\div$ 75 = 8·64. Balance, 3·36 feet. |

The balance Head of 3·36 feet is more than neutralised by the depth of the floor beyond the Head. This settles the sufficiency of the foundation from a statical point of view. The rest of the length of the masonry floor is determined by hydro-dynamical considerations, in which precedent alone can be followed, on which the empirical formulas already given in Chap. VII. are founded.

(21) The draw gates for the Undersluice are shown with the outer gate cambered on plan, an arrangement convenient for forming a watertight connection between the two, by a stanching rod or a series of narrow hinged flaps. The plan usually adopted is to place the gates back to back, *i.e.*, with their straight sides facing each other inside, as is shown in Fig. 2, Chap. VII.

Both grooves are carried down to the floor; this enables regulation with the top gate to be effected by lowering as well as raising it, and is preferable to stopping the inner groove at the crest of lower gate, as is sometimes the practice.

The crest of the top gate of the Undersluice is shown as 1 foot higher than F.S.L. in the canal. As a matter of fact, a head of 6 inches would more than suffice for the purpose of forcing full supply into the canal over the obstruction of the dwarf weir at the head.

The F.S. depth is 8 feet. Assuming a mean velocity of 3 feet per second, the canal discharge will be 8 × 3 = 24 cubic feet per second per foot run of width. Now, with a head of ·5 foot, the depth of film passing over the dwarf weir, which is submerged, will be 6½ feet. By reference to Table III., Series I., Chap. V., it will be seen that the discharge due to this depth of 6½ feet, with $H = \frac{1}{2}$, is 29·792 cubic feet per second, *i.e.*, more than is required, or if allowance be made for end contractions it will be reduced to just about what is necessary, viz., 24 cubic feet per second.

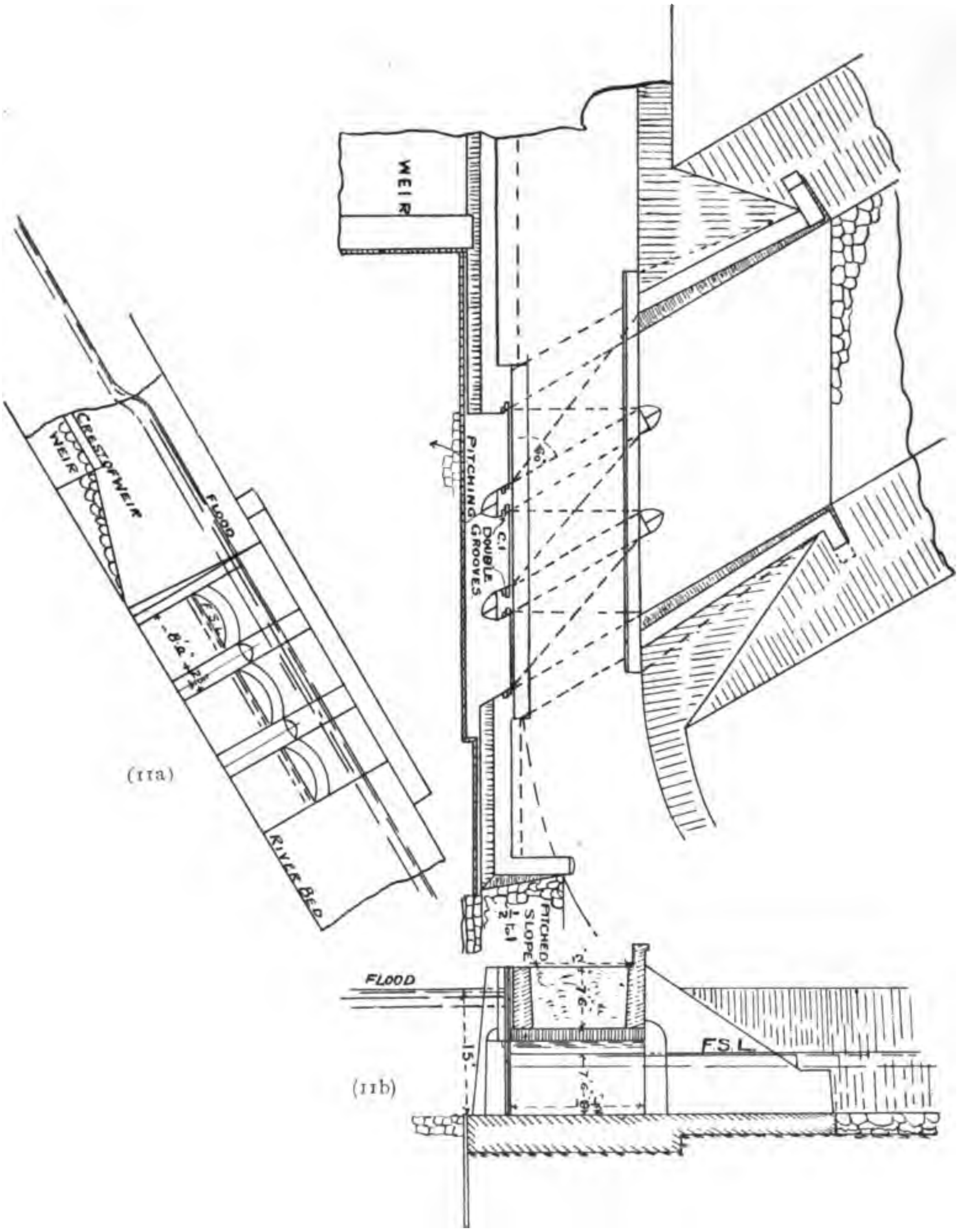
This investigation is useful as showing the advantage accruing from the adoption of a fall at the canal Head in cases where the canal bed level is either flush with or but slightly raised above that of the sill of the Undersluice. The diminution of the waterway of the entrance into the canal, which is obstructed by the dwarf weir, is compensated for by a comparatively very small increase in the head outside, and the raising of the sill at the entrance of the Head work provides depth outside the work for the temporary accumulation of deposit in suspension in the river, which would otherwise be swept into the canal. This deposit can be periodically scoured clear by the occasional opening of the Undersluices.

**(22)** Almost all canal Regulators take off at right angles to the axis of the stream, *i.e.*, generally parallel to the direction of the weir. As the course of the canal must be more or less parallel to the river this arrangement involves a large curve in the canal excavation close to the head work. This is objectionable as causing a diminution of the velocity of the current with consequent increased liability to the deposit of silt. The Kistna East Canal, Madras Presidency (Fig. 6d, Chap. VI.), takes off at a very sharp angle to the axis of the stream, probably in order to avoid a rocky hill or some such impediment. The difficulty in that case is got over by the Head Regulator (which is square) discharging into a large basin from which the canal takes out. A similar difficulty is provided for by the arrangement shown in Fig. 11a, Chap. VII., of the Laguna Head Works.

Without doubt it would be decidedly advantageous to construct skew heads at many canal head sites. The disadvantage which is inherent in a skew head is the widening of the opening involved and consequent increase in the length of the gates. Now, however, that anti-frictional rollers can be fitted to any gates, be they iron or wood, thus immensely reducing the power required to manipulate them, this objection vanishes altogether. There remains only the question of cost, the skew head being naturally somewhat more expensive. The author has constructed a skew head to a small canal, which answered perfectly, and its adoption improved the alignment of the canal, some heavy cutting being thereby avoided.

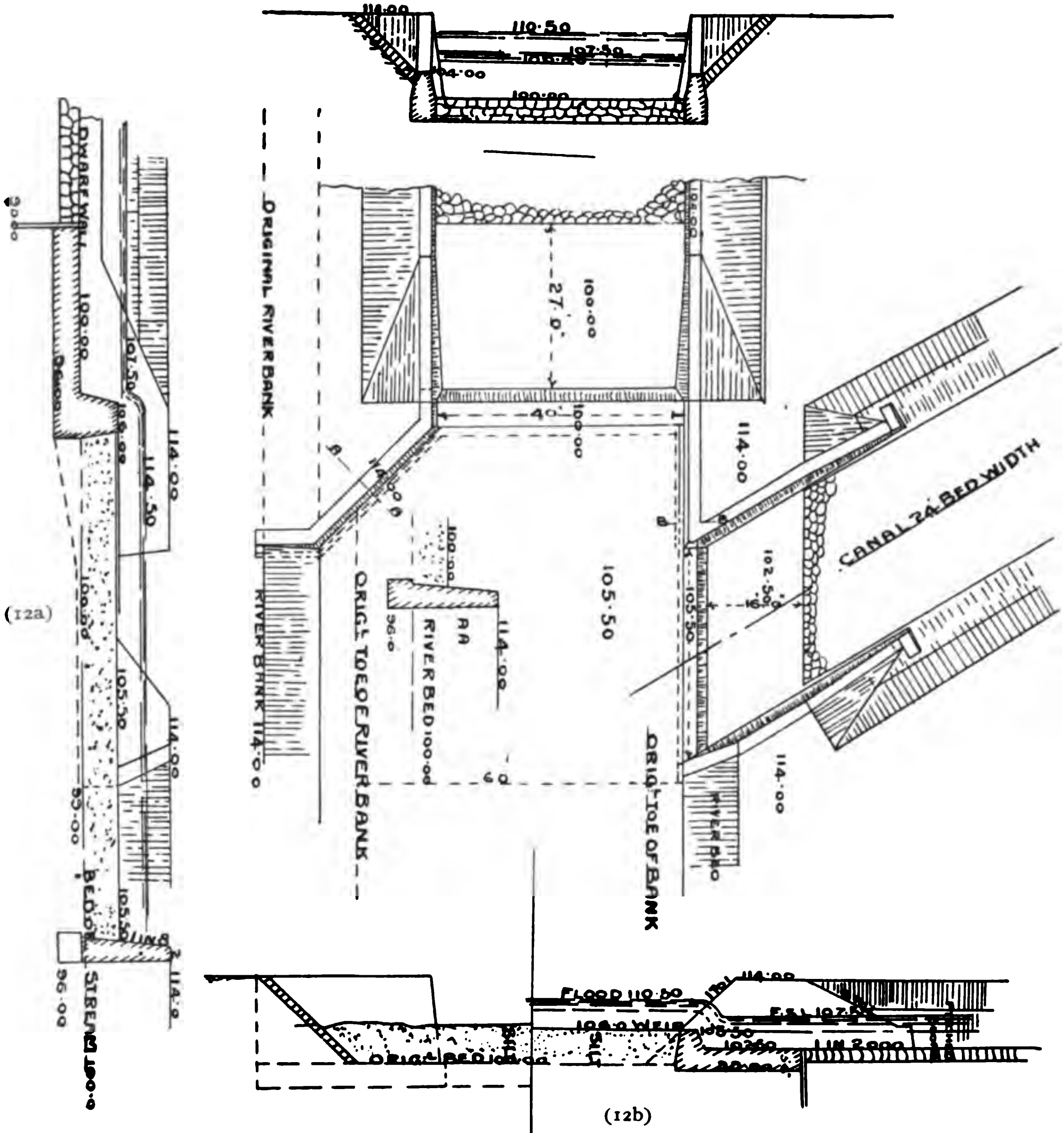
**(23)** In Fig. 11 a design for a skew head is exhibited. The angle of a skew is 60°, the same as was adopted in the small headwork just alluded to.

The canal is assumed to take off at river bed level; the weir is 6 feet high and the afflux 9 feet deep over the weir. A dwarf weir, 3 feet high (erroneously omitted in the figure), is built across the floor at the entrance. No Undersluices are adopted, as the river is supposed to be a small one of intermittent flow, and a scouring escape is provided lower down.



FIGS. 11, 11a, 11b.—Design for Skew Head.

The weir wall in the canal Head admits of silt deposit 3 feet deep in the canal without interference with the discharge of entry. The material thus



(12a)

(12b)

FIGS. 12, 12a, 12b.—Design for Open Canal Head.

deposited will form a steep bed slope. This can be either removed periodically or an escape scouring head can be constructed in solid ground lower down the canal, as has been done in this case. To make this escape effective, a

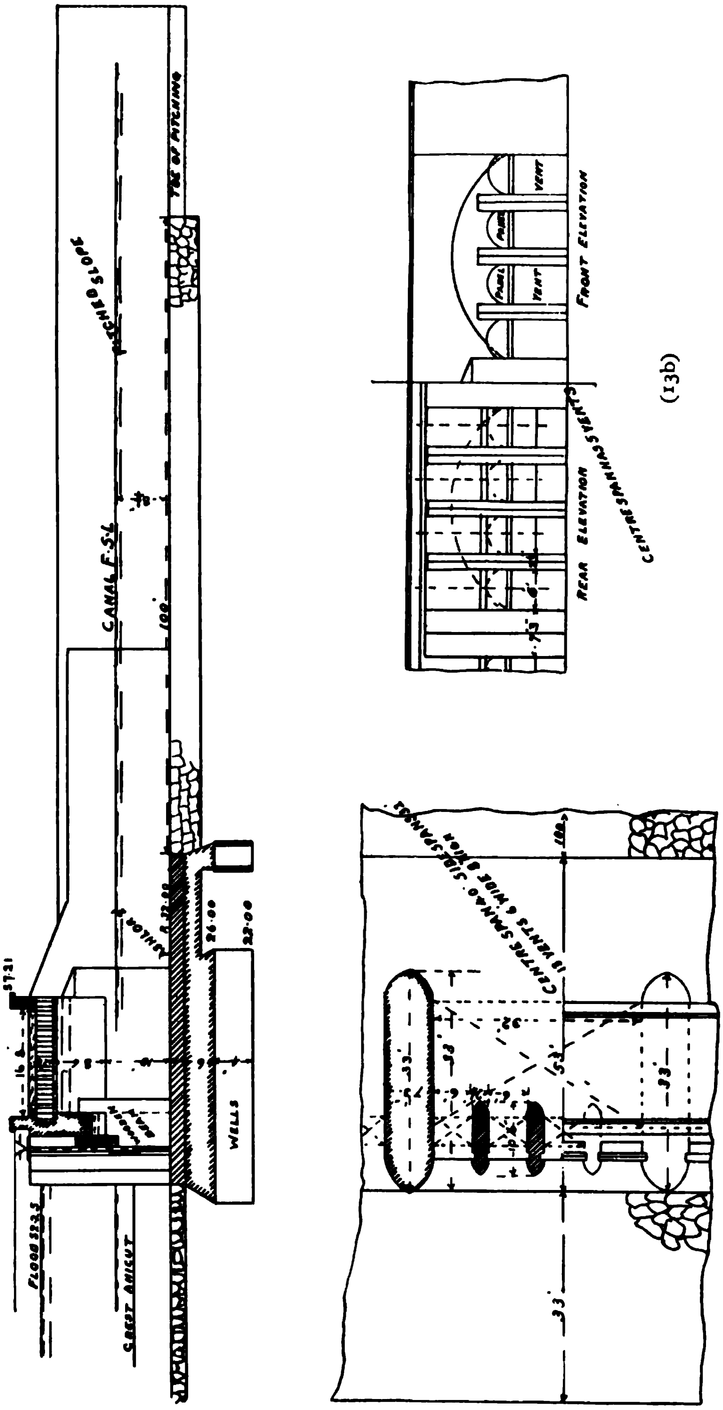
Regulator should be built across the canal just below the Escape Head. By this means, at intervals when the supply is available, the whole discharge can be passed down the escape channel, which will effectually scour out the silt deposited near the head work. The canal bed and banks will require protection by pitching up to the Escape Head.

(**24**) If the floods in the stream or river are not very high, a Regulator with gates at the canal Head can be dispensed with altogether, and the canal Head will then consist simply of an opening protected by masonry flank walls and floor. This arrangement is common in American Head Works (*vide* Figs. 18 and 19, Chap. XIII.).

A design on these lines was made for a canal leading out of an intermittently running stream in Burma (*vide* Fig. 12). Assuming the R.L. of the bed of the river to be 100·00, the crest of the river weir was fixed at 106, that of the Head canal weir at 105·50, or 6 inches lower, crest shutters 2 feet high being provided. The canal Head work took off at an angle of 58° and consisted merely of the weir wall, with a masonry floor flanked by two battering wings. The floor of the Head work and bed of the canal were at R.L. 102·50, or 3 feet below the canal weir crest and 4 feet below the river weir crest. The flood discharge of the stream was about 1,200 cubic feet per second, and the maximum discharge of the canal was 450 cubic feet with a depth of 5 feet. With 1 foot depth of water passing over the river weir, the canal would run about 2 feet deep, with a discharge of 120 cubic feet. With 2 feet over the river weir, the canal would run 4 feet deep with a discharge of 300 cubic feet, and with 3 feet over the weir 5 feet deep with a full discharge of 450 cubic feet. At about 300 yards below the canal Head, an escape was designed leading back into the river, with a skew regulating bridge across the canal just below it. This was for use when water was not required in the canal; the whole supply could then be turned down the escape, causing the silt deposited to be scoured out, the canal bed and banks being pitched for this length. The canal bed slope was 1 in 2,000.

The silt deposited can accumulate to a depth of 3 feet inside the Head without in any way blocking the waterway, which deposit could be periodically scoured out by occasionally passing the supply through the escape. This design, it is believed, will in a measure solve the difficulty of the gathering of as large a supply as possible from small intermittent streams, and also that of the prevention of the blocking of the waterway by silt deposit, which is always to be counted on, especially when there are no Undersluices provided. Shutters on the weir 2 feet deep would be an improvement.

(**25**) The majority of rivers and streams from which water for irrigation can be taken are not, as in the case of Upper India, large rivers with perennial flow, in many cases derived from the melting of snow on the Himalayas, but are intermittent in flow, coming down in occasional freshets during the rainy season, and running perfectly dry during most of the rest of the year. Such rivers are numerous in South Africa. It is a common

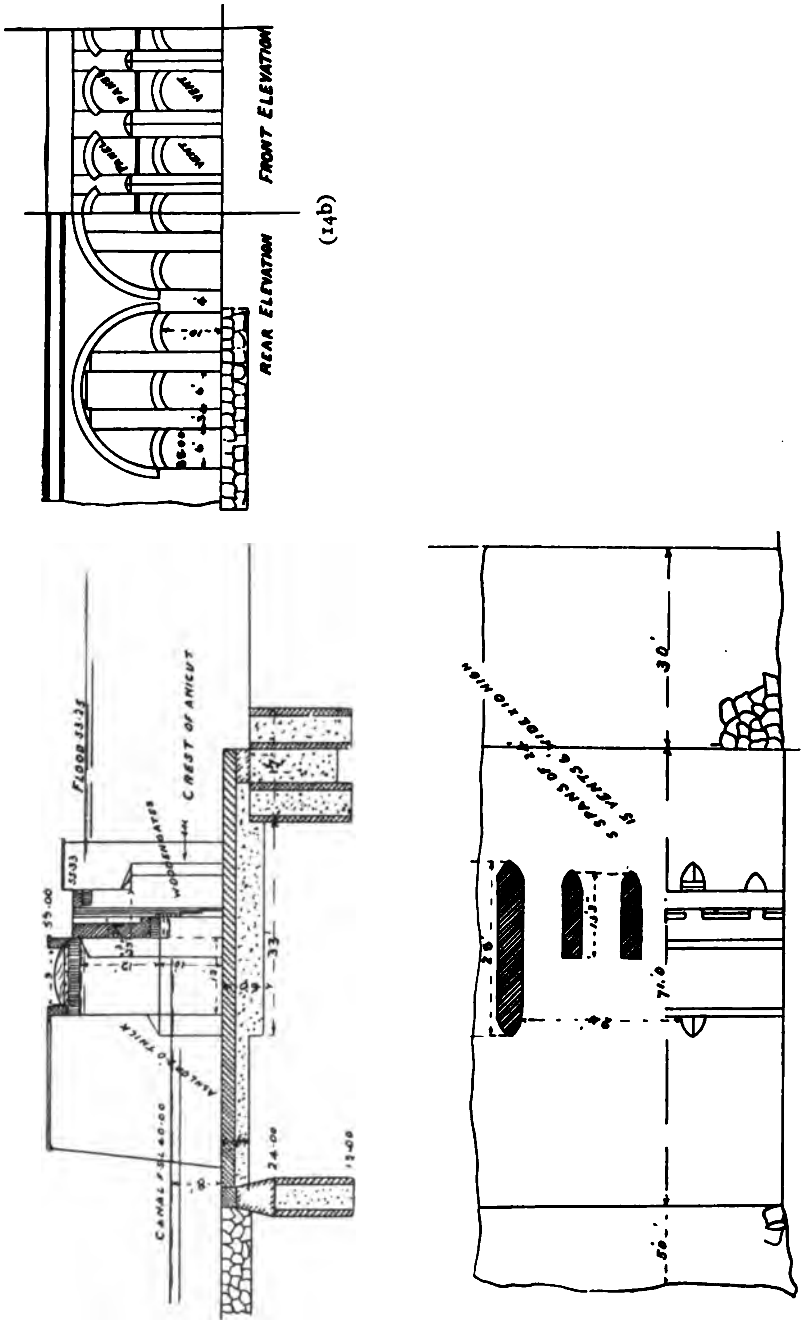


(13a)

(13b)

FIGS. 13, 13a, 13b.—Dauleshwiram Head Regulator.

error to suppose that small rivers that only bring down water to local rainfall are useless for irrigation purposes; such is by no means the case.



(14b)

(14a)

Figs. 14, 14a, 14b.—Babarlanka Head Regulator.

Where the local rainfall is insufficient to raise valuable crops such as rice or sugar cane, it can be supplemented with great advantage by irrigation from an intermittent stream. Besides which, these rivers often come down in flood when there is no local rainfall, a rainburst in the hills where their source lies causing a freshet. This water can be utilised either by direct irrigation from a canal or a combination of direct irrigation and storage. The author has constructed several works of this description in Upper Burma. First, the stream was dammed and water taken off by a canal; from this direct irrigation on fields took place, and surplus water was allowed to pass through the fields which were under a rice crop, and tail into another drainage line. This again was dammed and another small canal taken off. In this way several drainage systems were practically connected, the main stream tailing into a large tank, whence the surplus water over the waste weir found its way into other tanks and eventually into the original watercourse, which again was dammed and the procedure repeated. The fields themselves, particularly rice fields, which are banked by mud walls 12 inches high, will hold an immense deal of water, and consequently do not require a continuous supply; a freshet once a week or ten days would keep them well supplied with water. The same applies to other crops, but in a less degree.

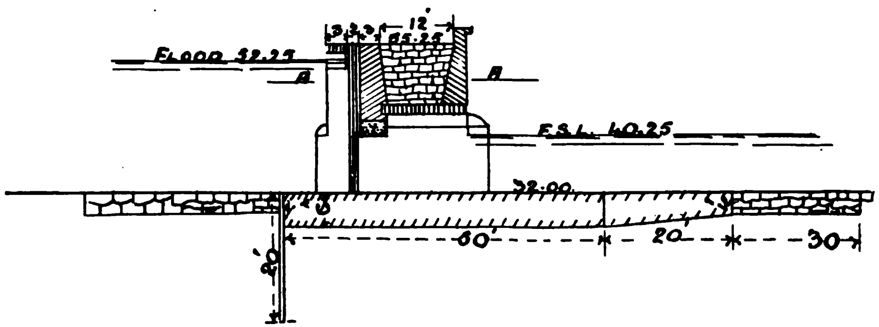
**(26)** The thickness and length of the floor in the examples given are designed on the assumption that the foundation is sand. As canals, if possible, invariably take off on the cutting side of a river, the Head Regulator is often entirely, and the Undersluices partially, founded on solid clay. In such cases no calculation is required for frictional stability of the floor, and the thickness of the latter beyond the Head work is given by the following formula: $\sqrt{H_1}+1$ ($H_1$ being Full Head). Thus in Fig. 10 $H_1 = 17\frac{1}{2}$; consequently the thickness of the floor will be $\sqrt{17{\cdot}5}+1 = 5{\cdot}2$ feet. The length will be $2H_1$ or $2 \times 17\frac{1}{2} = 35$ feet, counting from the grooves.

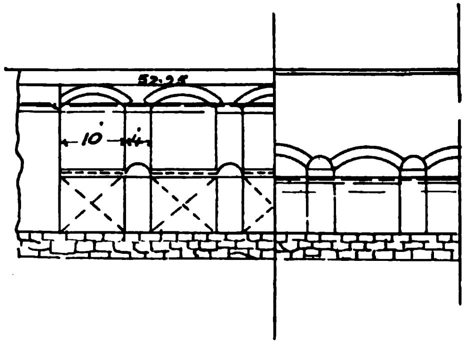
The floor hardly requires thickening under the Head work, and could be made, say, 6 feet thick here to distribute the weight. The thickness of piers in these works should be not under $\cdot 4S$, as a general rule (*vide* Chap. IV., para. 5).

**(27)** We will now give an example of the old style of canal Head Sluice, of which there are many examples in Upper India, as well as in Madras. The design consists of a large span bridge with the interior filled up with smaller piers and vaulting. The system, now that larger vents can be adopted, is quite obsolete, and at any time is a clumsy arrangement. Fig. 13 represents the Dauleshwiram, and Fig. 14 the Babarlanka Canal Head. The works are situated on either side of the Godaveri anicut. These plans are instructive mainly as regards the foundations.

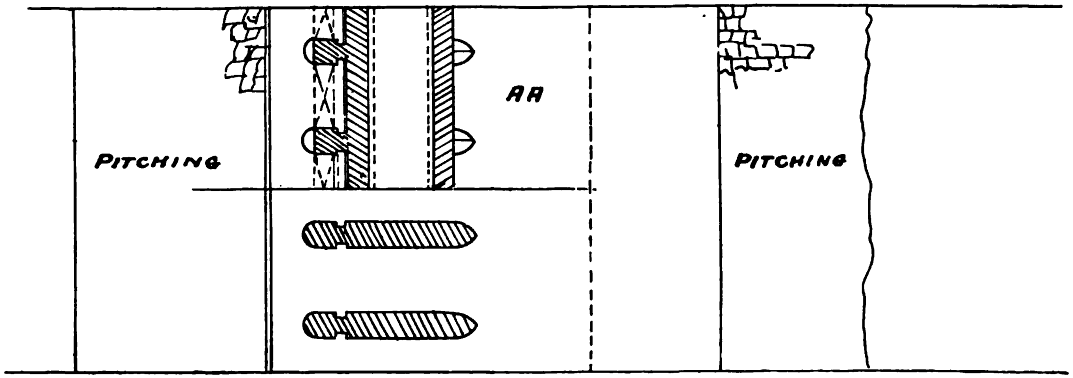
**(28)** In Fig. 15 is a revised design for the Dauleshwiram Head under the same conditions.

The U section is adopted with outside arch to carry the outer rail of the traveller. The vents, which are 10 feet × 8½ feet, are spanned on the river face, by rolled beams and concrete. Against the projecting flange of the





(15a)



(15b)

FIGS. 15, 15a, 15b.—Design for Head Regulator.

outer beam the upper draw gate nearly touches, the connection being made watertight by a horizontal stanching rod attached to the inner side of the top flange of the gate. Beyond the base of the breast wall the vaulting begins, the spring of the arches being placed higher than the upper sill of entrance vent. A raised sill, if adopted, would probably be of advantage in this as in most cases.

The coefficient of the River Godaveri is ·75, and the floor area has been designed accordingly, the Head being 20½ feet.

The estimated value of $Ap$ for the superstructure for a length of 14 feet is 9,462, dividing by 14, the quotient is 676 per foot run. The analysis will be as follows, the fore pitching assumed stanched with puddle :—

| | $A$ | | $p$ | $Ap$ | |
|---|---|---|---|---|---|
| Fore pitching . | 120 | × | ·8 | 96 | Head, 20·5. |
| Sheet piling. . | 200 | × | ·6 | 120 | |
| Superstructure . | — | | | 676 | |
| Floor, 6 feet deep | 300 | × | 1¼ | 375 | |

Total . . 1,267 ÷ 75 = 17 nearly. Balance, 3 feet.

The downward pressure at this point is 6 × 1¼ = 7½ feet.

Continued—

Floor, 5 feet deep, 100 × 1¼ = 125 ÷ 75 = 1·67. Balance, 1·33.

The length of pitching 4 feet deep required to neutralise the remaining head will be $\frac{1·33 \times 75}{4 \times ·8} = 31$ feet. The ratio $\frac{6}{H}$ in this case is 10, should be 12.

It will be noticed that the sectional area of the floor of this design is much less than that of Fig. 10, although the head is greater. This is accounted for by the higher specific gravity allowed for the superstructure and also for the floor in this design, the deeper sheet piling, and the fore pitching. In these works, the value of $l$ as co-factor of $Ap$ always requires attention.

(29) An example of a canal Head work built on boulder formation, viz., that of the Western Jumna Canal Head, is given in Fig. 16. In this case the sill is raised 4 feet above that of the Undersluice, the floor being built on a slope forming a rapid. On this sloping floor the piers of the work are built; a rapid of this description discharges just as much as a vertical drop.

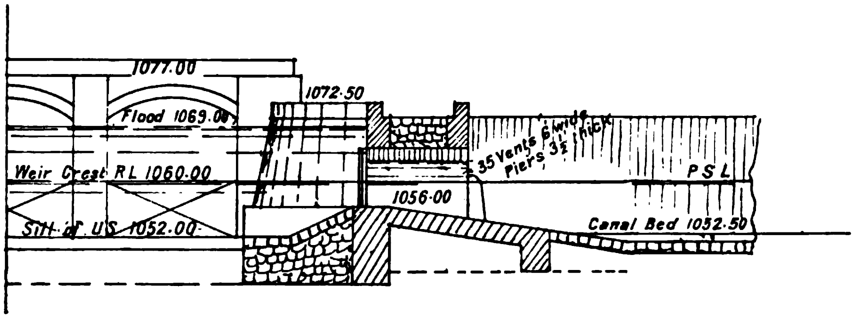


FIG. 16.—Western Jumna Canal Head.

There appears to be no advantage in this arrangement. The object of raising the sill was to keep boulders from being carried into the canal, the river passing them down over the weir and through the Undersluice at every freshet. The regulating bridge is of the usual U pattern, one peculiarity being that the piers are not provided with cut-water noses in front, but end flush with the face of the rest of the work. This arrangement is to obviate any trap for the accumulation of boulders or detritus in front of, or in the vents, and where there is a drop in the Head, as in this case, the absence of cut-waters is not so much felt, so that the arrangement, under the circumstances, is decidedly a good one. The vents, which are only 6 feet wide, are closed by wooden draw gates, a recess being formed by an upright beam built into the sill, behind which the gates work. This system of numerous narrow spans is now quite obsolete, the slow regulation being now superseded by wider spans, closed by double roller gates raised by a travelling winch.

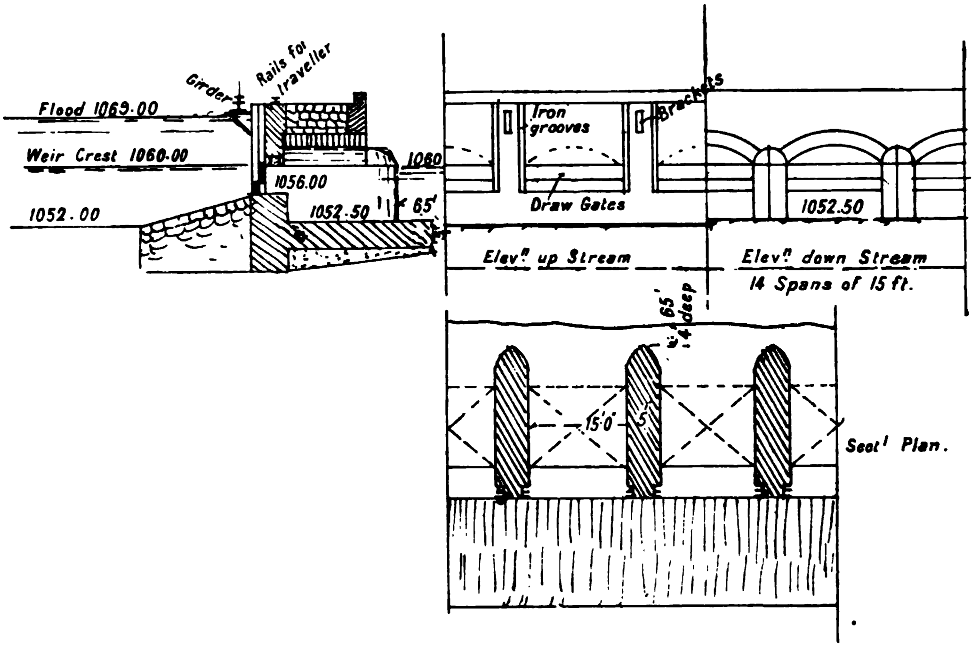


FIG. 17.—Alternative Design.

(30) An alternative design for this work on modern lines is given in Fig. 17. In this the general arrangement is retained, but the spans are increased from 6 feet to 15 feet, with 5 feet thick piers, *i.e.*, of a proportion of ·33*S*. As a great deal of the water pressure is taken off by the raised sill, the thickness of the piers can well be reduced below the excessive proportion of ·6*S* previously adopted, the piers being 3½ feet thick.

Iron grooves for a double set of draw gates are inserted at the square extremities of the piers. These gates will be manipulated by a travelling winch running on rails, one of which is laid on the parapet wall of the bridge, and the other is carried on a longitudinal girder, which on its part is supported at each pier by an iron or steel bracket bolted to the structure, the arrangement being identical with that in Fig. 10b. The arching over the vents stops short of the face of the work, and horizontal girders are substituted, so as to close the arch segment and provide an abutting surface for the top gate.

Plans of the weir and Undersluice of this Head work are given in Fig. 10, Chap. VII.

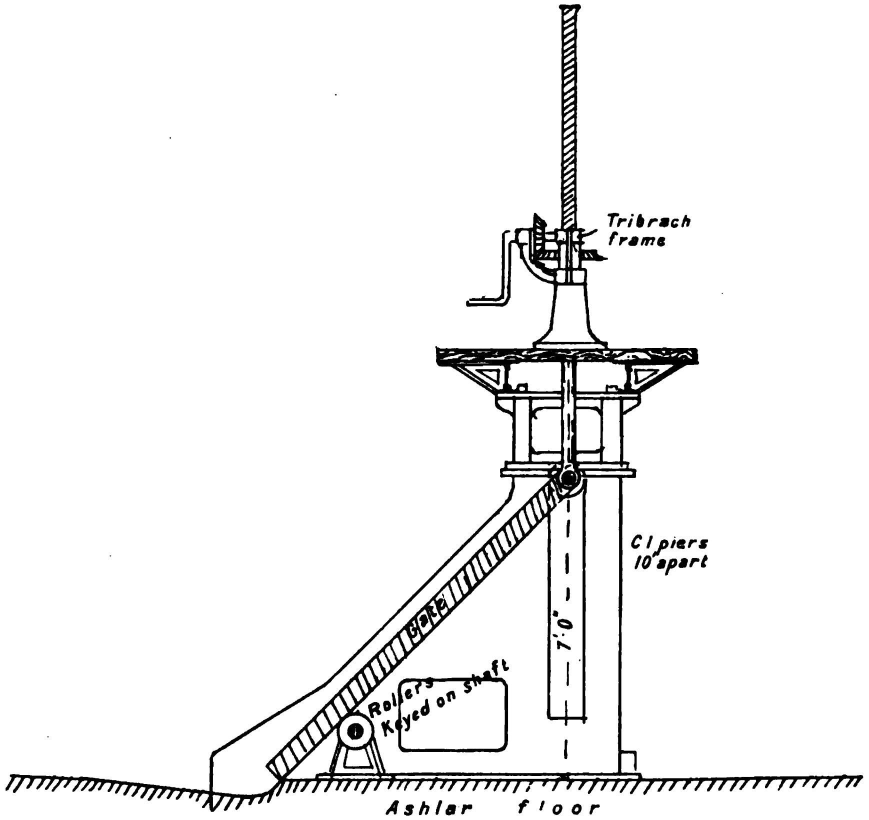


Fig. 18.—Goulburn Canal Regulator.

(31) Fig. 18 illustrates a remarkable and novel design for canal Regulating Head and gates, employed in the Goulburn Canal, Australia, which is given in "Irrigation Engineering" (Wilson). As will be seen, the gate, when closed, is inclined at 45°, and is attached at the centre to the end of a screw rod which moves it in a vertical direction. The other end rests loosely near the centre of pressure on four rollers carried on a shaft, which is supported by four fixed plummer blocks. The upper ends of the gate are carried on wheels which just fit in vertical grooves in the framework of the iron piers. The gate is thus forced to work vertically; the bottom end is consequently pushed downwards and outwards, eventually assuming a horizontal position when the sluice way is completely open.

By this ingenious disposition regulation can be effected with a minimum of applied force, and, in addition, the water is only drawn from the surface. This arrangement, however, would not work where silt is allowed to accumulate at the foot of the gate, and consequently would not answer for Head Regulators in ordinary cases.

The piers, as well as the abutments, are built of cast iron, and are 10 feet apart.

(**32**) The screw-lifting apparatus consists of a cast-iron standard provided with a separate head, carried on an oblong vertical frame of two arms joined at the head. In the space thus formed, the female screw head fits, to which is attached a horizontal bevel wheel. This bevel wheel is worked by a vertical pinion attached to handle shaft, which is carried by a third independent arm. The hollow screw head bears upwards on the frame head and downwards on the top surface of the lower part of the frame, hence does not require a separate thrust plate.

This arrangement, in common with all others where the female screw has the motive power, is subject to the great disadvantage of the screw rod being in one piece and rising above the platform and frame, and being thus exposed to dust or rain, besides being liable to be bent or damaged. As fully explained in Chap. XIV., where several examples of screw gearing are given, this fault can be remedied by having the motive power applied to the solid, not to the hollow, screw. The latter, which should be fixed to the head of a pipe, is prevented from revolving by being attached to the gate, as also to guides, and thus is forced up or downwards by the male screw, which travels inside the pipe. Thus the solid screw is effectually protected from dust and water, and nothing appears above the head of the standard.

This arrangement is obviously superior to any other, and markedly so where the lift is short, as the expensive solid steel screw need only be as long as the lift of the gate, whereas the long connection is formed by the pipe, which is better suited for compressive and tortional strain, and need not be more than an ordinary gas or water pipe having a hollow screw head fitted at one end.

The propriety of using cast iron frames for piers and abutments of regulators in lieu of masonry is entirely one of comparative cost, which varies immensely in different countries, and so cannot be settled by any hard and fast rules.

(**33**) In Fig. 19 we have an instructive example of quite recent American practice, viz., that of the Minidoka Canal Head in Idaho, a work that was still incomplete last year (1906). Most irrigation works in the United States have hitherto been constructed, in a large measure, of timber, so that the institution of permanent works of masonry is a comparatively new departure. Under these circumstances we should expect to find in these latest exponents of design some novelty, if not some improvement over the oriental types common to India and Egypt. A casual inspection of the plans of this work, and that illustrated later in Fig. 21, will, however, at once discover to the critical eye that not only no advance whatever has been made in principle or practice, but that old errors of design, long ago abandoned in oriental works, have been carefully reproduced. The principal of these improvements, largely influencing design, is the adoption of anti-friction rollers to draw gates, whereby larger vents can be used than were formerly practicable, which arrangement tends to economy of material, as well as to the possible avoidance of expensive screw gear for manipulating the gates.

In this work, the head of water to which the superstructure is exposed is very moderate, being only 14 feet, *i.e.*, much below what is usual in Indian canal Heads, consequently a light superstructure is all that is required to provide the weight necessary to oppose the overturning moment of the water pressure.

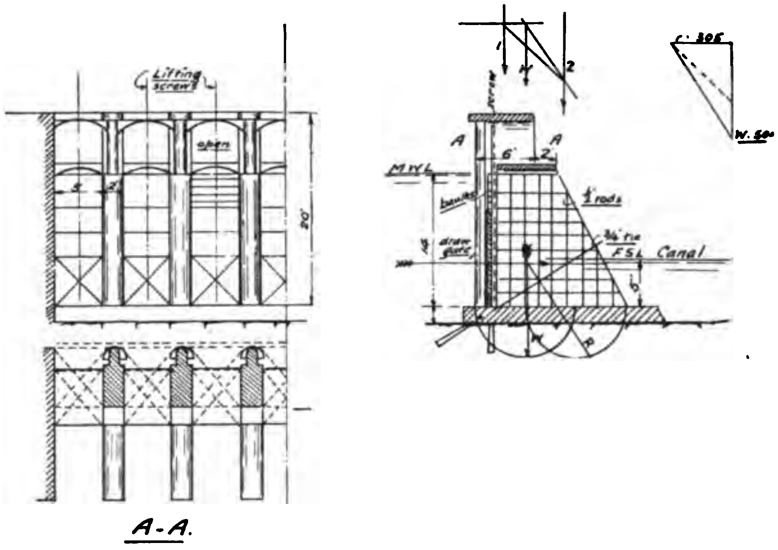


FIG. 19.—Minidoka Canal Head.

In the design the piers are 2 feet thick to spans of 5 feet, a proportion of ·4*S*. This proportion is suitable in cases where the head of water equals or exceeds 20 feet, but with a low head, as in the present instance, the proportional thickness undoubtedly errs on the side of solidity, and yet excess of solidity in the design of Indian irrigation works is what is generally found fault with by American critics.

Another point open to objection is that high level arches are adopted with no breast wall in front, consequently the onus of closure of the vents up to full flood level is thrown entirely on the draw gates and their superimposed baulks. The gates are therefore 10 feet deep in one piece, and the resulting friction due to the exposure of so large a surface to water pressure, even under moderate head, with the absence of anti-friction rollers, results in the necessary adoption of long screw rods of some 2½ or 3 inches diameter. The conditions, therefore, of the old Madras works, now long superseded by better methods, are reproduced. Again, the thick piers only 5 feet apart run right up to the full height in front of the work, though somewhat reduced on the rear, where a lower level vaulting is introduced, with the object, apparently, of enabling the manipulation of the top closure baulks to be effected, reserving the very narrow top platform for the screw standards and gear.

To prove the unnecessary solidity of the construction, the resultant line of pressure, together with the concomitant graphical diagrams, has been drawn on the profile in Fig. 19, with the result that its incidence on the base line of the piers lies too far within the middle third with regard to strict economical considerations. The pier bases therefore are subjected to a very moderate maximum pressure, estimated at under 1½ tons per square foot. In spite of this and the fact that the overturning moment is neutralised more than is at all requisite, we find that the piers are reinforced with a network of steel bars, all of which, as well as the anchor bars let into the rock, is absolutely superfluous and unnecessary.

The adoption of a single lifting gate is open to the serious objection that water can only be admitted at the bottom, except on occasions when the river level is over 10 feet above the floor, *i.e.*, on the comparatively few occasions when it passes over the waste weir. Double or treble draw gates running in separate grooves enable water to be admitted at the very top by lowering the upper gate behind the lower gate; in this way the lowest stratum of water is never disturbed, until absolutely necessary. Silt is thus allowed to deposit, instead of entering the canal head, and can be scoured away by Undersluices, sand sluices or other expedients.

The screws working the gates are operated by a worm and bevel gearing which is attached to a hollow or female screw head, held in the standards. This revolves and draws the long solid screw bar to which the gate is attached up and down as may be required. When the gates are raised these fourteen solid screws will project 10 feet above their standards into the air. This objectionable and primitive arrangement is easily avoidable, by the simple expedient of applying the motive power to the solid rod, while the female screw head is attached to the end of a pipe fixed to the gate, which pipe does not revolve, and inside which the solid screw is protected from wet or rust, nothing appearing above the standard.

**(34)** In Fig. 20 an attempt has been made to provide a more economical and more efficient design. The spans are increased to 10 feet and the piers made .3*S* or 3 feet thick, which is ample under the conditions of head. A pair of roller gates closes the entrance to the vents, which are 10 feet high. This considerable excess over the F.S.L. in the canal is adopted in order to draw water always from as high a level as possible. Above this level a simple breast wall carried by narrow arches closes the upper part, the piers stopping short at spring level. A 6 feet way is formed by iron brackets planked over, at the rear of the breast wall. An outer arch is provided for the outer rail of a travelling winch.

These travelling winches are all provided with double winding drums, one over each groove, so that the gates are hauled up, or lowered by chains attached to hooked bars projecting upwards and sheltered in recesses in the grooves, so that they can easily be fished for and engaged from above, in case the chains themselves are not drawn to the surface and hooked there ready for attachment to the winch drums. For details, *vide* Min. Pro. Inst. C.E., Vol. CLVIII., 26.

On the section, Fig. 20, the upper gate is shown partly lowered, admitting water.

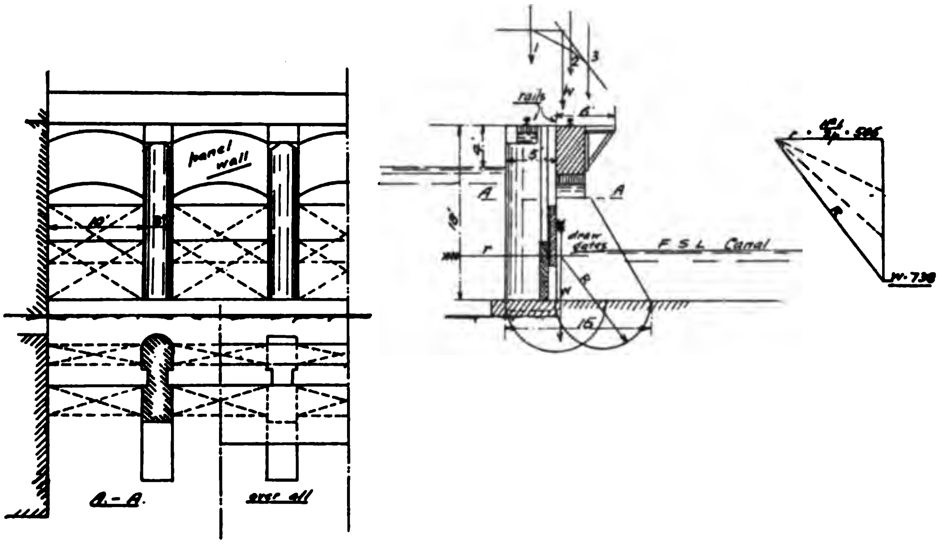


Fig. 20.—Alternative Design.

The comparative quantities of masonry or concrete in either work are as follows:—

Fig. 19: Fourteen spans of 5 feet, 2 feet piers, with reinforcement.
Quantity in 7 feet lengths, 480 cubic feet.
Per foot run, 69 cubic feet.
Fig. 20: Seven spans of 10 feet, 3 feet piers, without reinforcement.
Quantity in 13 feet lengths, 790 cubic feet.
Per foot run, 61 cubic feet.

(**35**) In Fig. 21 is another example of a recent work in America, viz., the Folsom Canal Head Regulator. This work consists of three spans of 16 feet, the piers being 6 feet thick, or a proportion of ·4*S* nearly. The head is much greater than in the former example, being 32 feet. In this design the depressed arch is very properly made use of, its springing being 10 feet above floor. The rest of the work is built up solid for an average width of 20 feet. The area of each draw gate is 16 × 14 feet, which is enormous considering the great head of water. The gates are of wood, not provided with anti-friction rollers, so that very exceptional arrangements have to be made to manipulate them. This is effected by the adoption of hydraulic jacks, one to each gate. These jacks have plungers 14 feet in length fastened to the rear of each gate, and are operated by a head of water provided from a power house.

It is considered that the section, which is very massive, could be economised with advantage, retaining existing conditions of span, etc., by reducing the weight of the solid mass filling and lengthening the piers. The adoption of a U section with side breast walls filled between with any heavy material, would effect great economy in masonry, without much diminishing the weight; a narrower top width with a battered inner breast wall being also adopted, as shown by a dotted line on the section of Fig. 21. As, however, in this canal in particular, the prevention of silt deposit is an urgent matter, it is deemed that a complete remodelling of the design allowing of water being drawn at different levels, much as was done in the alternative design for the Betwa Head (Fig. 8), would be more suitable. The exceptional size of the vents will also well bear reduction, in view of easy manipulation of the gates by ordinary means.

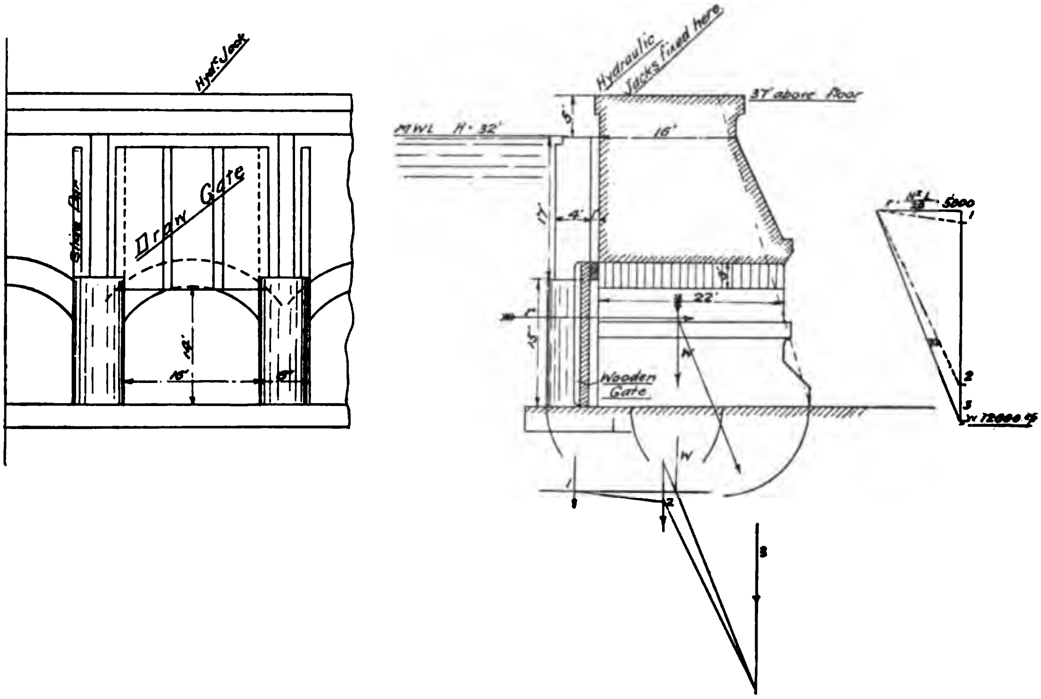


FIG. 21.—Folsom Canal Head.

(**36**) In Fig. 22 the design is remodelled on these lines, the spans are reduced to 10 feet with piers 4 feet thick, the number being increased from three spans to five.

Three tiers of gates are provided in three sets of grooves with outside arches to accommodate the travelling winch. The U section is adopted with arches depressed to 10 feet above springing. These alterations result in very substantial economy, as below:—

Fig. 21: three spans of 16 feet, 6 feet piers.
Quantity in 22 feet lengths, 12,600 cubic feet. Per foot run, 573 cubic feet.
Fig. 22: five spans of 10 feet, 4 feet piers.
Quantity in 14 feet lengths, 4,200 cubic feet. Per foot run, 300 cubic feet.

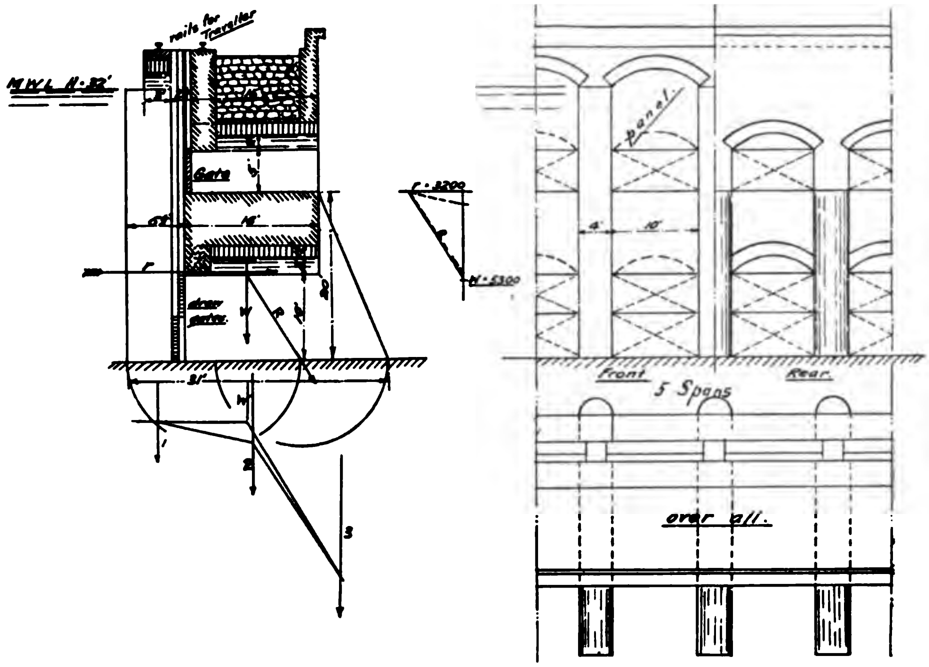


FIG. 22.—Alternative Design.

This is much to the advantage of the alternative design in point of view of economy, while the efficiency as regards regulation and prevention of silt entry is undoubtedly enhanced to a considerable degree.

In both these works, viz., Figs. 19 and 21, the same arrangement is adopted for the gates, which slide on flat iron bars let into the face of a recess in the piers, the single or double U-shaped cast iron grooves, usually adopted in modern practice in India and Egypt, not being employed. The rounded cut-waters also do not extend above the height of the gates, so that there is no semblance of groove above this level. This arrangement would not suit where gates are lifted at each end, not at the centre, the groove being necessary to protect the lifting chains from the current.

A site plan of the Folsom Canal Head Works is given in Fig. 18, Chap. XIII.

# CHAPTER IX.

## CANAL FALLS.

(1) WHEN the slope of the ground surface exceeds that of the bed of a canal, vertical falls have necessarily to be constructed.

In many cases these drops in the bed are of frequent occurrence, and thus form a by no means inconsiderable item in the cost of the whole work. It is, consequently, very desirable that in their design strict economy should be combined with efficiency.

Fifty or sixty years ago, when the first great irrigation works, such as the Ganges, Jumna, Bari Doab, Godaveri and Kistna Canals, were constructed, hydraulic science was in a less advanced state than at the present time, and, in addition, successful precedent for guidance in design hardly existed. Consequently, owing to the prevalence of erroneous ideas regarding the destructive effect of a mass of falling water, it was deemed essential to avoid a direct overfall; and thus the drop walls were designed on the profile of an ogee curve. The object sought to be attained was that of diverting the falling current into a horizontal direction, and by this means obviating any direct vertical impact in the floor. Experience, however, soon proved that, although the supposed destructive action was successfully diverted from the floor of the fall, the effect of the increased velocity engendered below the work was most injurious to the banks and bed of the canal.

Up to a comparatively recent date the crests of the drop walls of canal falls were built raised above the canal bed level of the upper reach. This effected a vertical contraction of the area of the waterway at the fall itself, proportionate to the increased velocity of the current, and thus the velocity of the overfall was restrained to that admissible in the earthen channel above; that is to say, the velocity of approach was restrained to that of the supply channel.

For example: Supposing the water in the upper reach to be 6 feet deep, with a mean velocity of 3 feet per second, the mean discharge would be 18 cubic feet per second per foot run.

If the crest of the weir were built level with the canal bed, the length of the crest being that of the average width of the channel, then the depth of film passing over will be such as would suffice to give a discharge of 18 cubic feet per second. Now the formula for discharge over a weir (free overfall), usually adopted, is $3{\cdot}333d^{\frac{3}{2}}$; if this be equated with 18 cubic feet, or else the result obtained from tables, the value of $d$ will be found to be 3 feet. The velocity of approach will then be $\frac{18}{3} = 6$ cubic feet per second. Such a velocity would be most destructive to the canal bed and banks, and interfere with the water level and regimen of the canal for a mile or more above the fall.

To obviate this, if the crest of the drop wall be raised 3 feet above the canal bed, a depth of film of 3 feet will, as we have seen, exactly pass the discharge. Hence the surface slope of the water in the canal up stream will in no way be affected, nor the mean velocity of the channel interfered with. Similar effect can also be produced by lateral contraction.

This simple matter of raising the crest of the canal weir would answer perfectly were the water level in the canal constant. Such, however, is not the case.

This difficulty of varying supply is practically overcome by the device of so-called "notch" falls, which have proved successful to such a degree as to be now universally adopted.

In these, the crest of the drop wall is raised up to Full Supply Level, and the part thus raised above the canal level is pierced by a series of trapezoidal openings, termed notches, through which the water passes. The reduction in sectional area is thus effected entirely by lateral, not by vertical, contraction, as the base of the notch is at the level of the canal up stream. The subject of the discharge of canal notch falls has already been dealt with in par. 26, Chap. V., where the coefficient of discharge of free and submerged notch falls is determined. The Tables numbered IV. and V. in the same chapter supply lists of discharges per foot run, under various conditions of depth and velocity of approach, whether free or submerged.

(**2**) It now remains to undertake the practical design of the works in detail.

In Fig. 1, are given an elevation plan and section of a notch opening said to be used in the Chenab Canal, while in Figs. 1a and 1b are similar views of a notch fall actually constructed on the same canal. These drawings are derived from "Irrigation Works in India." The extreme elaboration of detail shown in the diagrams in Fig. 1 does not seem to be carried out in practice in Figs. 1a and 1b; in the latter case the profile of the notch sides on plan is simply a segment of a circle.

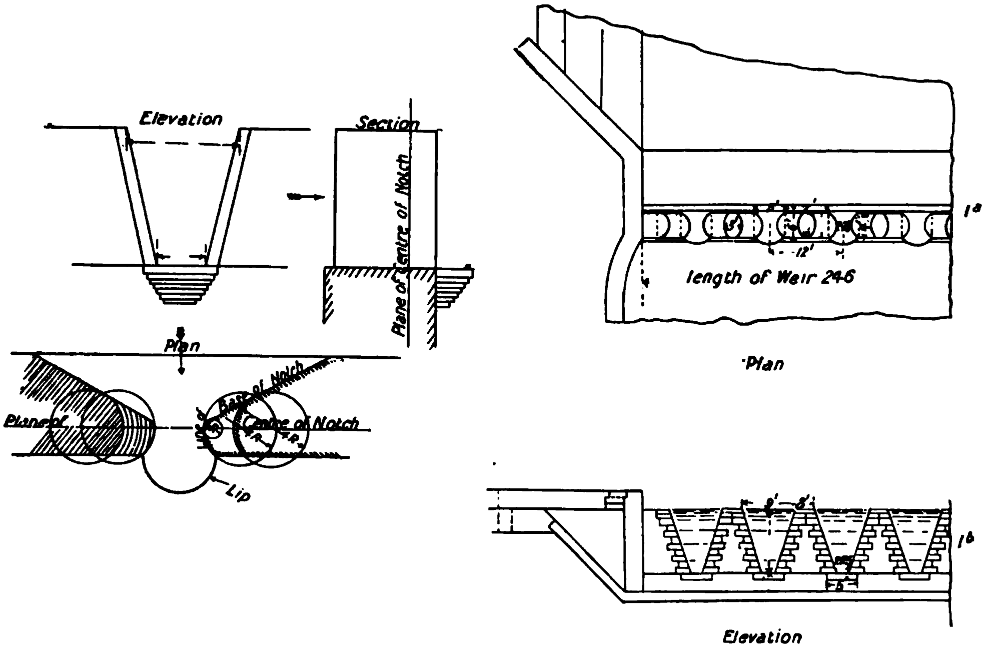
The plan (Fig. 1) would not answer practically, on account of the space required between each notch opening to admit of the great splay given to the sides of the inner face. Such elaboration would undoubtedly conduce to raising the value of the coefficient of discharge; but considering that the trapezoidal notch is after all exactly suitable to the discharge at two levels only, and the coefficient adopted is but approximate, such extreme refinement in the design of the outline appears out of place.

(**3**) The first point to be determined is the length to be given to the fall between abutments, and the number of notches it is advisable to adopt.

This length should not ever exceed the bed width of the canal up stream, and may well be less, up to a limit of, say, ⅞ of the bed width.

With regard to the number of notches, this point depends mainly upon the top width given to the trapezoidal openings. This top width should clearly bear some proportion to the depth of the notch, *i.e.*, to the depth of full supply. This proportion will be greater in a submerged, than in a free overfall, and likewise be in the inverse ratio of the velocity of approach. A safe rule to lay down is that the top width should never exceed the depth, and may vary from $\frac{3}{4}\,d$ to $d$.

For example: In Figs. 1a and 1b the fall is submerged, the drop being only 3 feet with a depth of film of 10 feet, and a slow velocity of approach. In this case the top width is $\cdot 9d$.



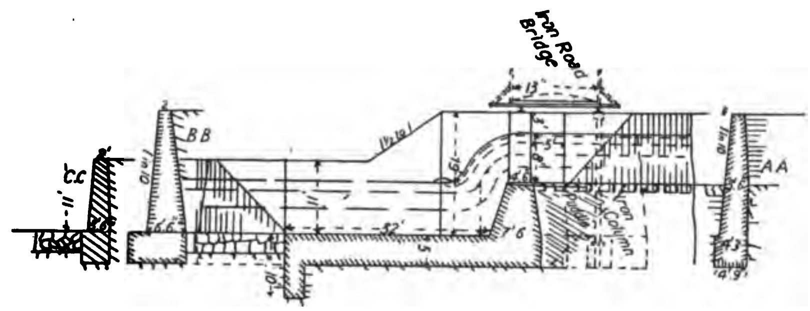
FIGS. 1, 1a, 1b.

(4) The thickness of notch piers should not be less than half the depth, and may be made more if they have to carry a superstructure, as is sometimes the case.

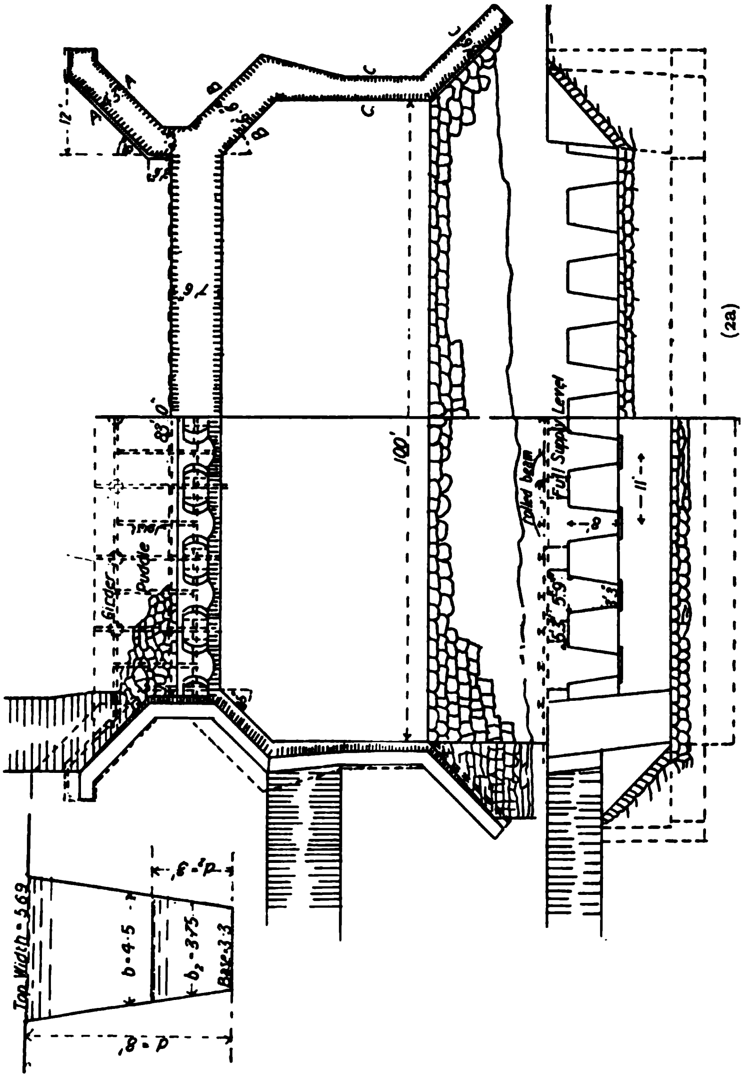
The top length of the piers should not, for purposes of stability, be made less than their thickness, particularly in cases where the piers are carried higher than F.S.L., so that the fall can be crossed by planks laid on their summits clear of the water.

(5) A practical example of the method of calculating the number and dimensions of notches will now be given.

The design in Fig. 2 is based on an assumed canal discharge of 2,500 cubic feet with a mean velocity of 3 feet per second, and a full supply depth of 8 feet (free overfall). To produce these conditions the slope of the canal bed will be about ·18 per 1,000 feet.



(2b)



(2a)

FIGS. 2, 2a, 2b.—Design of Notch Fall.

The area of the waterway will be $\frac{Q}{V}$ or $\frac{2,500}{3} = 833$ square feet.

The width of the canal bed, with side slopes one to one will also be $\frac{A}{d} - d = \frac{833}{8} - 8 = 96$ feet. The value of $R$ is $\frac{A}{WP} = \frac{A}{b + 2d\sqrt{2}} = \frac{833}{96 + 22}$ $= 7{\cdot}06$ nearly.

Now with regard to the number of notches. The overfall being free, and the assumed velocity of approach above the average, it can be safely assumed that the top width of each notch will not exceed $\frac{3}{4}d$, or 6 feet. We have seen that the length of the pier crests should not be less than half the depth of water, or 4 feet, and as the piers are intended to carry a light bridge, the crest length might well be increased to 5 feet.

The spacing of each notch will then be 11 feet. This will allow of eight notches, or a length of 88 feet between abutments, which is 8 feet less than the bed width of the canal. As, however, the abutments will be designed with battering faces, the actual length at base of the notch piers has been made 85 feet.

(6) Having tentatively decided on the number of notches, the discharge which each has to accommodate will be $\frac{2,500}{8} = 312$ cubic feet nearly.

If reference is made to Table IV., Chap. V., it will be seen that the discharge per foot run with $d = 8$ and $V = 3$, is 69·44 cubic feet per second. The required half width of each notch opening ($b$) will then be $\frac{312}{69{\cdot}44} = 4.49$ feet. This is the value of $b$, whatever may be the slope of the sides.

(7) The half width of the whole opening being thus ascertained, we have now to fix the same for another lower depth of water.

If a notch were designed to discharge precisely the proper amount at every change of level in the surface of the canal water in the upper reach, it would take an ovoidal outline similar to the profile of an egg with both ends truncated, the thick end uppermost. As, however, it has been proved that a trapezoidal-shaped opening correctly calculated to discharge the full supply, and that due to one lower depth, will answer all practical needs, this refinement need not be entertained.

Orifices of this ovoidal section could easily be provided, were cast iron piers adopted.

(8) To revert to the lower depth, it is found convenient to place this level below what is usually the lower supply in the canal.

In Upper India most canals run at two levels, the higher when water is most in demand, *i.e.*, in the hot season, when the rivers rise owing to the melting of the snows in the Himalayas, and again a lower supply in the winter months ; and this double supply level is what obtains in most canals.

Owing, however, to a variety of causes, the fluctuations of level are often considerable, so that it is advisable to adopt a value for $d_2$ below that of the lower supply level $d_1$.

This value should be a little over half the lower depth or $\frac{d_1}{2}$, or under half the full depth, *i.e.*, $\frac{d}{2}$.

In the case we are considering, the lower supply level $d_1$ is assumed at 5 feet in depth, but the value of $d_2$ will be taken as 3 feet.

**(9)** It now remains to calculate the discharge of the canal at this depth of 3 feet.

In this case $A = 99 \times 3 = 297$ square feet, $R = \frac{A}{WP} = \frac{297}{104{\cdot}4} = 2{\cdot}84$, $S$ is given as ·18 per 1,000, or $\frac{1}{5{,}555}$.

We have now to estimate the mean velocity from the formula $100\,c\,\sqrt{RS} = V$. Omitting $c$, the expression will become

$$V = \sqrt{2{\cdot}84 \times \frac{10{,}000}{5{,}555}} = \sqrt{\frac{5{,}540}{1{,}111}} = \sqrt{5{\cdot}11} = 2{\cdot}26,$$

If $S$ per 1,000 be considered, the equation will be $V = \sqrt{2{\cdot}84 \times 10 \times {\cdot}18} = \sqrt{5{\cdot}11} = 2{\cdot}26$ (*vide* par. 38, Chap. V.).

The value of the coefficient $c$, suitable to one of $R = 2{\cdot}84$ and of $S = {\cdot}18$ per 1,000, obtained from Table XII., Part IV., of the "Hydraulic Manual," is ·7 nearly. The actual mean velocity will then be the previously obtained value of $2{\cdot}26 \times {\cdot}7 = 1{\cdot}582$ feet per second, and the discharge of the canal at the lower depth of 3 feet will be $A\,V$, or $297 \times {\cdot}582 = 475{\cdot}2$ cubic feet per second.

**(10)** Again dividing by the number of notches, the discharge through each must be $\frac{475{\cdot}2}{8} = 59{\cdot}4$ cubic feet. The half width $b_2$ is obtained, as previously, by reference to Table IV., Chap. V. In this we find that the discharge per foot run, with $d = 3$ and $V = 1\frac{1}{2}$, is 15·853 cubic feet per second. $b_2$ will therefore $= \frac{59{\cdot}4}{15{\cdot}85} = 3{\cdot}75$ feet.

**(11)** Having thus obtained the widths of the notch openings at two known levels, viz., at $\frac{d}{2}$ and $\frac{d_2}{2}$, we are now in a position to calculate the top and bottom widths.

The vertical distance apart of the two widths $b$ and $b_2$, are $\frac{d}{2} - \frac{d_2}{2}$. The ratio of widening will then be $2\,\frac{b - b_2}{d - d_2}$. Let these horizontal and vertical differences be designated $m$ and $n$. Then the expression becomes $2\frac{m}{n}$. Now the top of the opening is distant $\frac{d}{2}$ above the position of $b$, and the base $\frac{d_2}{2}$ below that of $b_2$, consequently the top width will be $\left(\frac{d}{2} \times 2\frac{m}{n}\right) + b$, or $(b + d)\left(\frac{m}{n}\right)$ (1). Similarly the base width will be $(b_2 - d_2)\left(\frac{m}{n}\right)$ (2).

The value of $m$ is $4{\cdot}49 - 3{\cdot}75 = {\cdot}74$.
That of $n$ is $8 - 3 = 5{\cdot}0$.

Therefore $\frac{m}{n} = \frac{{\cdot}74}{5} = {\cdot}15$, and transposing these values in the expressions (1) and (2) we have

top width $= 4{\cdot}49 + (8 \times {\cdot}15) = 5{\cdot}69$ feet and
base „ $= 3{\cdot}75 - (3 \times {\cdot}15) = 3{\cdot}3$ feet.

The average of both widths should naturally equal $b$, *i.e.*, $\frac{8{\cdot}99}{2} = 4{\cdot}49$.

The size of the notch is made 5 feet 9 inches at top and 3 feet 3 inches at base. With 11 feet centres, the top length of the piers will be 11 — 5 feet 9 inches = 5 feet 3 inches.

**(12)** Supposing the fall to be not free but submerged below the ⅔ limit, *i.e.*, the drop in the canal surface to be 2 feet only.

In such case, with full supply, the discharge with $d = 8$, $V = 3$ and $H = 2$, as in Table V., is found to be 63·15 cubic feet per foot run; the value of $b$ will then be $\frac{312}{63{\cdot}15} = 4{\cdot}94$ feet instead of the 4·49 in the free overfall. The lower depth $b_2$ will not be submerged over $\frac{2}{3}\, d_2$, consequently its value will remain as before at 3·75 feet.

Then $m = 4{\cdot}94 - 3{\cdot}75 = 1{\cdot}19$
and $n$ as before $= 5{\cdot}0$
Whence $\frac{m}{n} = \frac{1{\cdot}19}{5} = {\cdot}24$ and the
Top width $= 4{\cdot}94 + (8 \times {\cdot}24) = 6{\cdot}86$
Base „ $= 3{\cdot}75 - (3 \times {\cdot}24) = 3{\cdot}03$

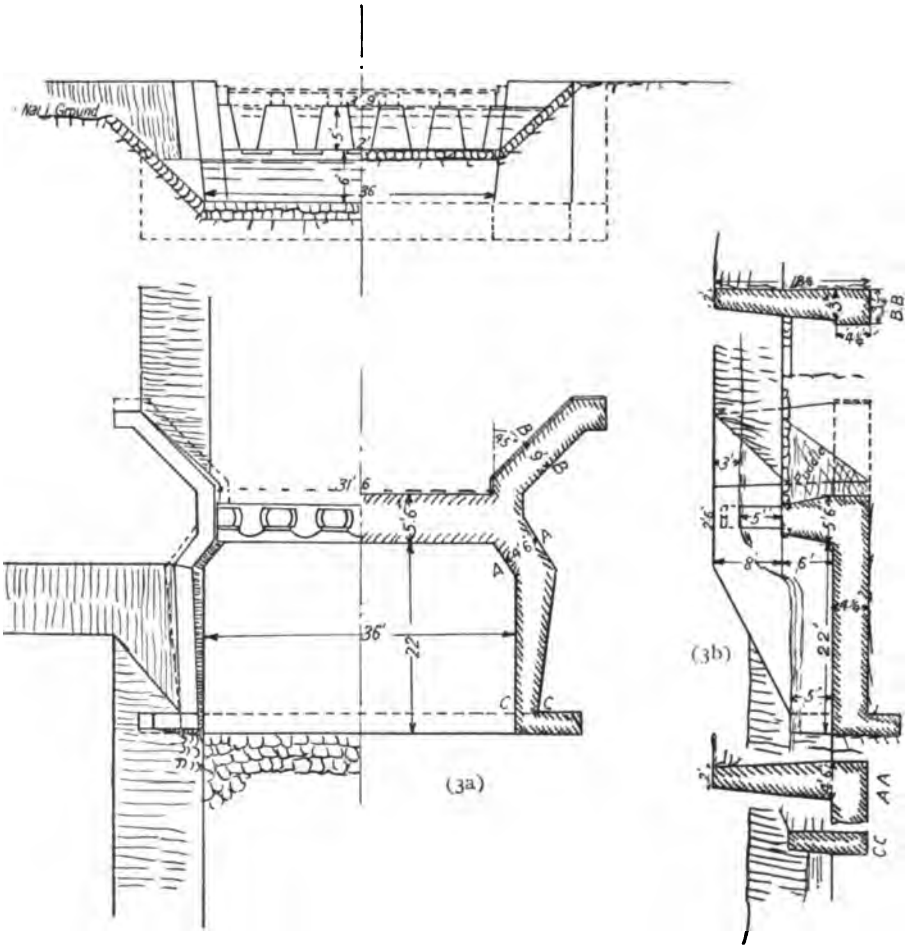
Total . . 9·89. Mean, $4{\cdot}94 = b$.

With this, the yaw of the opening will be noticeably greater.

**(13)** Fig. 3 is another design on similar lines to the last, but under differing conditions. Here $Q$ is 500 feet and $V$ $2\frac{1}{2}$ feet, $d = 5$ feet and the canal bed width 35 feet, and $A = 200$ and $R = 4$. There being five notches (*vide post*), the full discharge through each will be $\frac{500}{5} = 100$ cubic feet per second. By reference to Table IV., Chap. V., the discharge per foot run due to $d = 5$ and $V = 2\frac{1}{2}$ is 34·4 cubic feet per second; therefore $b$ will $= \frac{100}{34{\cdot}4} = 2{\cdot}9$.

(**14**) The lower level will be taken at a depth of 2 feet. Here $A = (35 + 2) \times 2 = 74$, $R = \frac{74}{40{\cdot}6} = 1{\cdot}8$.

The discharge can be obtained with close approximation without having to calculate $S$, by using the proportional ratio of $V$ in either case. Thus



FIGS. 3, 3a, 3b.—Design of Notch Falls.

$v : V :: 100c\sqrt{r} :: 100C\sqrt{R}$, *i.e.*, $:: c\sqrt{r} : C\sqrt{R}$. In this case $r = 1{\cdot}8$, $R = 4$ and $V = 2{\cdot}5$, whence $v = \frac{2{\cdot}5 \times 1{\cdot}34}{2} \times \frac{c}{C}$.

If $S^{\circ}$ in 1,000 be assumed anywhere near its right value, the true values of $C$ and of $c$ can be obtained with close approximation from Table XII., Part IV., "Hydraulic Manual." Let the slope be assumed at ·2 per 1,000. Then with $R = 4$, $C$ will be ·755, and with $r = 1{\cdot}8$, $c$ will be ·627; $b$ then will be $\frac{2{\cdot}5 \times 1{\cdot}34 \times {\cdot}627}{2 \times {\cdot}755} = \frac{2{\cdot}1}{1{\cdot}51} = 1{\cdot}4$ nearly.

To check this, the actual bed slope will now be found, using the coefficient ·755 as before, $V = 100c\sqrt{RS}$.

Transposing and substituting values,

$$\sqrt{S} = \frac{2\cdot5}{200 \times \cdot755} \therefore S = \frac{6\cdot25}{40\cdot000 \times (\cdot755)^2} = \frac{1}{6400 \times \cdot57} = \frac{1}{3648},$$ whence $S^{\circ}$ in 1,000 = ·28 nearly.

(15) Now a slope of ·2 per 1,000 was assumed, but if the slope be taken as ·3, the corresponding value of $C$, with $R = 4$, is ·754 against the previously adopted value of ·755. In the case of $c$ the coefficient is as ·631 to ·627. So that any alteration in the already obtained value of $V$ is unnecessary. If the divergence in the ratio of the two old coefficents with that of the two new, *i.e.*, $\frac{c}{C}$, were sensibly different, the calculation would have to be gone over again, assuming a more approximately correct value for $S$ (*vide* par. 42, Chap. V.

(16) Kutter's coefficients are undoubtedly troublesome to manipulate even with the help of tables. Their value is now being slowly but surely discredited. In Egypt Bazin's coefficients are still used, never having been superseded by Kutter's.

These general questions of coefficients could easily be set to rest once for all by experiments on a large scale instituted by the Indian Irrigation Department. The only known systematic officially conducted experiments are those by Col. Cunningham on the velocity of the water section in a channel, which, though doubtless valuable from an academic point of view are of little, if any, practical value. The want of proper coefficients applicable to the discharge of water, on a large scale, through orifices and overfalls particularly, is also a crying evil which could easily be settled by proper arrangements instituted by authority, with a practical man in charge who knows exactly what is wanted and how to get at it.

(17) To resume the calculations, we have already seen that for the lower depth of $d_2$ the value of $A$ is 74 feet, the discharge will then be $VA$, or $1\cdot4 \times 74 = 104$ cubic feet per second nearly. This divided by the number of notches, viz., 5, the quotient 20·8 is the required discharge at the lower level. By reference to Table IV., Chap. V., the discharge per foot run for a value of $d = 2$ and $V = 1\frac{1}{2}$ is 8·74 cubic feet per second. The lower width $b_2$ will then be $\frac{20\cdot8}{8\cdot74} = 2\cdot4$.

The corresponding width ($b$) at half the depth ot the notch has already been ascertained to be 2·9, whence $m = \cdot5$ and $n = 3$ and the half ratio $\frac{m}{n}$ will be $\frac{\cdot5}{3} = \cdot17$.

The top width will then be $(b+d)\left(\frac{m}{n}\right) = (2·9 + 5) \times ·17 = 3·75$, and the

base width ,, ,, $(b_2 - d_2)\left(\frac{m}{n}\right) = (2·4 - 2) \times ·17 = 2·06$

| | | |
|---|---|---|
| Total | . . | 5·81 |
| Mean | . . | $2·9 = b$ |

**(18)** Supposing the drop were 3 feet instead of 6 feet as shown in Fig. 3*b*, the tail water would then reach 2 feet above the base of the notch, when full supply was running. This, however, would not affect the discharge, the weir being submerged only $\frac{2}{5}$ of the depth. Not until it is over $\frac{2}{3}$ drowned is it treated as a submerged fall and reference made to Table V., Chap. V., instead of Table IV. (*vide* par. 17, Chap. V.).

**(19)** In the design Fig. 3, the length between abutments has been taken as $31\frac{1}{2}$ feet, *i.e.*, $3\frac{1}{2}$ feet less than the bed width, which is 35 feet. The depth of water being 5 feet, 4 feet was estimated for the top width of the notch openings and $\frac{d}{2}$ or $2\frac{1}{2}$ feet for that of the piers, making the notches, five in number, $6\frac{1}{2}$ feet apart. Total length measured at notch crest of $32\frac{1}{2}$ feet. The batter of the abutments, which is one in ten, will reduce this at base of notch piers by 1 foot, being 5 feet of height at one in ten.

**(20)** The designs in Figs. 2 and 3 are similar in all respects as regards arrangement of parts. The weir wall is of the "hybrid" type of section (*vide* par. 19, Chap. III.). The base being $\frac{H_1}{\rho}$ or in Fig. 2, $\frac{4}{9} \times 16 = 7·1$ feet (made $7\frac{1}{2}$ feet); in Fig. 3, $\frac{4}{9} \times 11 = 5$ feet (made 5 feet 6 inches).

The top width is made 4 feet 6 inches wide in Fig. 2, *i.e.*, $\sqrt{H} + \frac{1}{2}$; and in Fig. 3, 3·6 or $\sqrt{H} + \frac{1}{4}$. The extra width is required for the accommodation of the notch piers, the thickness of which in both cases is $\frac{d}{2}$.

The length of the floor in both cases is $2H$, according to the rule given in par. 36, Chap. IV.

The thickness of the floor is likewise $\sqrt{H} + 1$.

With regard to the width of the floor, it is made in both cases somewhat greater than the canal bed width, viz., about one-thirtieth wider.

The floor is terminated by a shallow curtain wall, beyond which it is protected by pitching.

**(21)** The abutments, as well as the wings, are all battered at one in ten. The abutments are continued up stream for a short distance parallel to the axis of the work, forming the commencements of a pair of level crested splay walls with end returns, of the type mentioned in pars. 64—69, Chap. I.

These not only form an excellent guide to the current on its contraction at the fall, but with the assistance of the level crested portion of the dog-legged down-stream wings, afford a wide level connecting bank and an approach to the bridge way across the fall.

**(22)** The down-stream wings on plan are of the so-called "dog-legged" pattern, having a re-entering angle (*vide* par. 39, Chap. I.). They start splaying outwards from the weir wall until the floor half-widths are reached. This portion is level crested. When the floor widening is thus effected, the direction of the base of the wings runs parallel to the axis of the work. In Fig. 2 the first portion has a sloping crest of two to one, corresponding with the drop in the canal bank; when this is overcome, it continues in the same direction at the level of the down-stream canal bank to the termination of the floor, when another splayed return forms a junction with the canal bank side slopes.

**(23)** In Fig. 3 the arrangement is somewhat different. Here the wall crests, beyond the re-entering angles, slope down at two to one till they reach the lower water level, when they make direct returns into the canal side slopes. The commencement of the sloped crests, and with them the re-entering angles on plan, are fixed with regard to space being left for the return walls within the length of the floor, which latter, as we have seen, is $2H_1$.

**(24)** The sections of the wings are strictly in accordance with the rules formulated in Chap. I. The sections at $AA$ in Fig. 2 and at $BB$ in Fig. 3 of the up-stream wings are of walls under partial earth pressure. The earth pressure exists only as far as the up-stream canal bed, below this level no additional external force is applied, so that the base at the upper canal level is made $\cdot 4H - 1$ wide, and below that point the back is carried down vertical but the face continues battered, as above ground; in addition a footing is provided near the base; the one in ten batter having at this depth to be thus supplemented to ensure the line of pressure falling within the base.

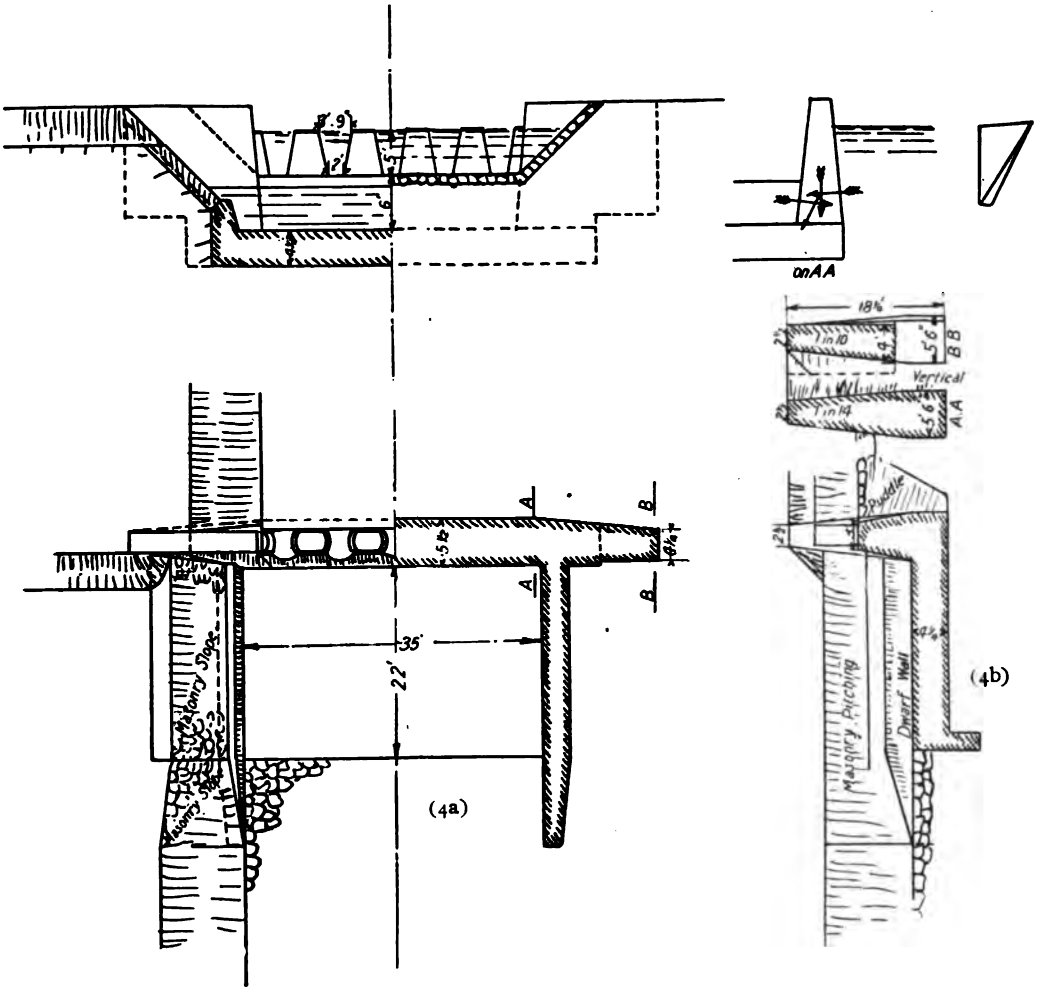
This important matter which is so often neglected has received considerable attention in Chap. I. (*vide* pars. 45—53).

**(25)** In Figs. 2, arrangements are shown for a light bridge across the fall. In this the notch piers are utilised to support square pillars of brick in cement, which carry an I beam A second longitudinal is bolted to iron columns which are founded on masonry blocks situated at the general foundation level, and these two are spanned by cross joists projecting well beyond the longitudinal beams, which in turn are covered with planking. Parapet rail standards are bolted to the projecting part of the joists, and thus a 13 feet wide roadway is provided.

The level of the platform is a little above that of the banks on either side, and the connection can be formed by a short slope.

An arrangement of this kind is equally effective and vastly more economical than the heavy masonry over-bridges which are such a common feature in old canal falls and which are perpetuated to the present day, as will be noticed later with reference to the Jamrao Canal Fall in Fig. 5.

In Fig. 3, a simple narrow footbridge is provided by planks laid on pillars built on the notch piers.



FIGS. 4, 4a, 4b.—Notch Fall, Wingless Type.

(**26**) We now come to an altogether different type of construction for a fall, which is illustrated in Fig. 4. The conditions are identical with those in Fig. 3, and the weir wall and notch piers are exactly similar. The principle of the construction is to effect economy in the flank defences of the work by causing the earth to support the protective walls, instead of *vice versâ*, so that masonry pitched slopes take the place of the down-stream flank retaining walls. Up stream there are no water wings whatever. Their place is taken by a horizontal and vertical prolongation of the weir wall, forming single direct return flank walls on each side of the weir, the up-stream canal side slopes running right up to them.

Down stream, the width of the floor is terminated on each side by a dwarf flank wall 3 feet high, which runs out with level crest to the end of the floor, whence the crest dips down at a slope of two to one to the down-stream canal bed level.

Above this dwarf wall, a masonry slope rests on the side of the earth cutting at a slope of one to one, and is carried on thus to the toe of the dwarf wall when it merges with the dry-pitched canal slopes.

The raised flank continuations of the weir wall have their foundations stepped up.

These walls at their commencement are subjected to lateral water, as well as earth pressure, and so the section gradually diminishes in width till, at their ends, the earth pressure from the up-stream side becomes very much reduced.

This is explained by the half-elevations and plans and the cross-sections on *AA* and *BB*.

(**27**) Excluding the weir wall and the visible portion of the floor, which is common to both Figs. 3 and 4, the quantities of masonry in the flank defences are as follows:—

| | |
|---|---|
| Fig. 3 . . . | 4,450 cubic feet. |
| Fig. 4 . . . | 2,210 „ |

This shows the great economy effected in the wingless design. The deeper the fall the greater would be the saving effected.

In a very wide fall, however, of 200 to over 400 feet in length, the cost of the flank defences form so small a proportion of the total quantity of work, that the economy effected in percentage of the whole, would not be very considerable.

Where falls are narrow, deep and numerous, this type of design is eminently suitable, not only on canals, but in reservoir by-washes, canal escapes, distributary falls, etc. The author has constructed several across rivers. One of these is illustrated in Fig. 24, Chap. I., and described in para. 69 of the same chapter.

(**28**) Fig. 5 is an example of a recently constructed canal fall on the Jamrao Canal, and is instructive as showing how a too close adherence to obsolete types results in a quite considerable waste of money.

The plans are taken from the Min. Pro. Inst. C.E., Vol. CLVII. p. 278.

The drop in the water surface is 6 feet, whereas that in the canal bed is 7 feet, the depth of full supply up-stream being 7½ feet, while that down stream is 8½ feet.

This shows that a change of bed slope takes place at the fall, the slope down stream being flatter than that obtaining above the work.

(**29**) With the sole exception of the notches, the fall is a more or less close copy of the old and now obsolete Ganges Canal types.

In the first place, a water cushion of no less than 6 feet in depth is provided for a drop of nearly the same amount, and that not of a concentrated waterfall, but one split into jets 11 feet apart. The provision of any water cushion to a notch fall is deemed quite unnecessary, if the floor is properly constructed.

**(30)** A second point in the design, which is very open to objection, is the combination of an overbridge with the fall. This is also a relict of old times. Except for reasons of economy, there can be no possible object in combining these two works. It is exceedingly unlikely that a road-crossing for traffic purposes is required at the exact site of a fall, and even if it were so, a very much more economical plan would be to build the bridge clear of the other work.

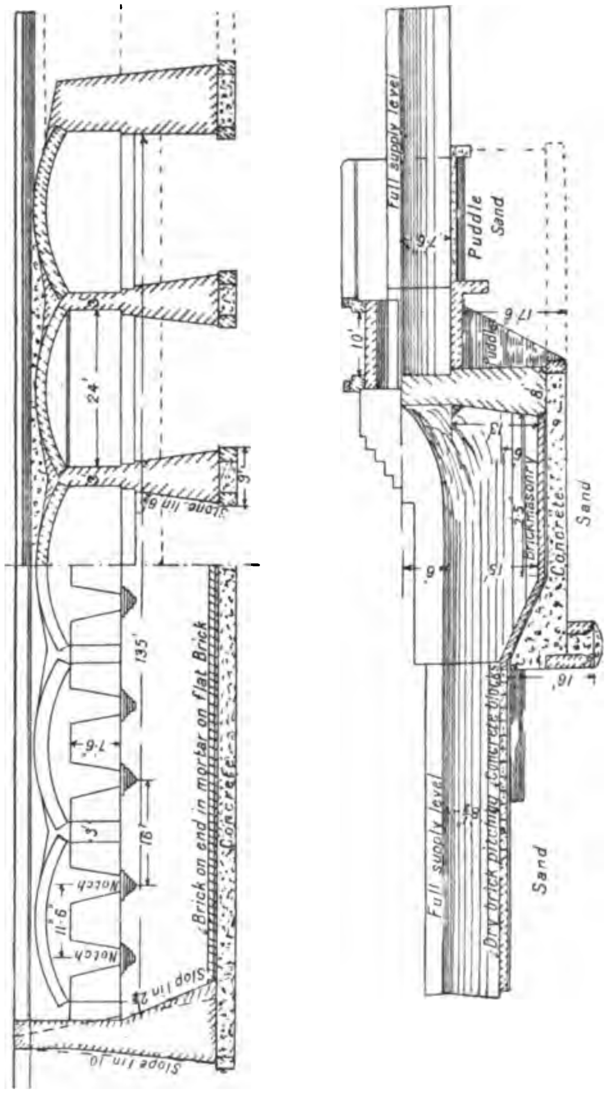


FIG. 5.—Jamrao Canal Fall.

From inspection of the drawing it is evident that this combination is productive, not of economy, but of the very reverse.

The bridge consists of wide spans of 24 feet. This in itself is an uneconomical arrangement, as high wide spans are only of practical use in a navigation canal. The foundations of the piers have to be carried right down to the base of that of the water cushion, *i.e.*, to a depth of 18½ feet below canal bed level. Not more than a foot or two in width of the whole bridge receives any support from the weir wall, so that the combination is clearly only productive of waste. Were the bridge built as a separate work, it would, or should, consist of moderate spans of, say, 10 feet, with short piers 8 feet high only, resting on a floor platform which need not be more than 3½ feet thick, enclosed between two lines of sheet piles. This construction would cost about half that of the present structure. In addition to all this, the presence of the bridge piers on its crest necessitates the lengthening of the weir by at least 40 feet, a further wholly gratuitous waste of material.

Another point involving a further unnecessary lengthening of the weir is the excessive length of the notch piers, which measure 6 feet long at the crest, whereas 4½ feet or 5 feet would have been ample. As there are ten notches, this would add another 15 feet to the length of the weir wall, making it altogether about 55 feet longer than is necessary.

(**31**) If the design were re-cast it would closely resemble Fig. 2, as the conditions of depth of water and drop in bed are not far different.

The main difference would consist in the number of notches and consequent length of the weir to suit whatever is the bed width of the Naza, and in the thickness and length of the floor, the former of which should be increased by the application of the formula for thickness suitable to sand foundations, viz., $\sqrt{H_1} + 1\frac{1}{2}$, and the latter increased to $3\,H_1$ for similar reasons.

Further, the sufficiency of the floor to resist the pressure of a head of water on a pervious substratum should be tested, and the floor section altered, or added to, as may be requisite, as has been done in cases put forward in the three previous chapters.

(**32**) It will be an interesting study to effect this analysis, taking Fig. 2 as model, but modified as required.

With canal running full supply the difference of levels, or $H$, will be 6 feet and the weir wall will be entirely submerged and half its weight absorbed by flotation. On the other hand, if the water in the upper reach were at bed level, that in the lower reach will be empty; the head will then be 1 foot more, viz., 7 feet, the weir wall, however, will not be subject to flotation. By a rough calculation it is found that the difference between the weir wall being submerged or not, represents a neutralisation of head of ·5 feet only, so that the effect of the 7 feet head will be the greater of the two by half a foot. These conditions will then be adopted.

(**33**) The differences in the design in Fig. 2 and that suitable for Fig. 5 consists in the following: The drop of the floor is 7 feet instead of 8 feet in the former, and the value of $H_1$, *i.e.*, height of surface of upper reach above lower floor is 14½ feet, not 16 feet. Consequently the section of the weir wall will present some difference. Further, the specific gravity of the Jamrao work is taken as 2, against 2¼ of that in Fig. 2.

The weir base will be $\frac{H_1}{p} = \frac{14{\cdot}5}{2} = 7{\cdot}2$ feet (par. 29, Chap. III.).

The thickness of the floor will be $\sqrt{H_1} + 1\frac{1}{2} = 5{\cdot}3$, and its length $3\,H_1$ or 43·5 (made 45 feet) (par. 20, Chap. IV.).

(**34**) In the following analysis $C = 100$, $H = 7$:—

| | A | p | Ap |
|---|---|---|---|
| Rear puddle (submerged) . . | 72 × | 1 = | 72 |
| Notches (free) . . . . | 15 × | 2 = | 30 |
| Weir wall . . . . | 42 × | 2 = | 84 |
| „ „ submerged . . | 37 × | 1 = | 37 |
| Total to toe of weir wall | | | 223 ÷ 100 = 2·23. |
| | | | Balance, 4·77 feet. |

The counteracting downward pressure is that of 5·3 feet of submerged masonry at specific gravity equal to unity, or 5·3 feet.

This point is therefore safe. To continue—

| | A | p | |
|---|---|---|---|
| Floor . . | 238 × | 1 = | 238 ÷ 100 = 2·38. Balance, 2.39 feet. |

We thus have 2·39 feet unbalanced, to be disposed of by the pitching beyond the floor. This will involve a greater length of pitching than is desirable; the length required, with pitching 4 feet deep, will be (by the formula $L = \frac{CH}{dp}$) = 75 feet nearly (*vide* par. 31, Chap. VI.).

(**35**) To reduce this to reasonable limits, sheet piling will have to be introduced, either at the commencement or the termination of the floor. The former is proposed for this class of work. We will thus suppose a line inserted in rear of the weir wall 16 feet in depth below base of work. Its value as regards frictional stability will be $16^2 \times {\cdot}5 = 128$. This divided by 100, the quotient is 1·28 feet of head neutralised. This will diminish the unbalanced head at end of floor from 2·39 to 1·1 feet.

The length of pitching required will then be $\frac{100 \times 1{\cdot}1}{4 \times {\cdot}8} = 24$ feet, a more reasonable figure. The revised section on these lines is given in Fig. 6 with this difference, that the floor is made 5 feet 6 inches deep on account of the low specific gravity of the material employed. If, owing to the friable nature of the soil, it would appear desirable to extend the length of the pitching beyond this amount, it could be done and the depth of the sheet piling correspondingly reduced. The value of $l$ (par. 19, Chap. VI.) works out $= 18\,H$.

The revised section on the lines indicated above is given in Fig. 6.

**(36)** The advantage of possessing definite rules for guidance applicable to the dimensions of every part of a structure, as also a method of testing the design with reference to frictional stability, will be made apparent by the foregoing paragraphs.

Facility of method combined with certainty and precision are indispensable aids to successful design, and the author's endeavour throughout this work is to smooth the rough paths, and render the designing of works an easy as well as a certain matter.

**(37)** Before proceeding to another subject, it will be well to notice the section of the flank wing shown in the elevation in Fig. 5.

It is unnecessarily thick at the top, 4 feet being quite twice the proper width. At weir crest level it is 5 feet wide, which is insufficient: it should be over 6 feet. The face line below this point suddenly begins to batter outwards at 1 in $2\frac{1}{2}$, giving a final base width of 13 feet. The total height is 30 feet. With a face batter of one in ten, the correct base width would be $\cdot 4\,H - 1$, or 11 feet, with a crest width of 2 feet. The section thus designed will have an area 195 square feet. The actual section has one of 198 square feet, so that as regards area there is not much to choose, but the revised section will be much more stable, the re-entering angle in the middle of the face being a bad feature. The object gained in throwing the toe of the wing forward and thus diminishing the waterway of the pool at the side is not apparent. On the contrary, any diminution of the waterway in the pool, except at a corner, is distinctly to be deprecated. The proper section for this wall is shown dotted on the drawing.

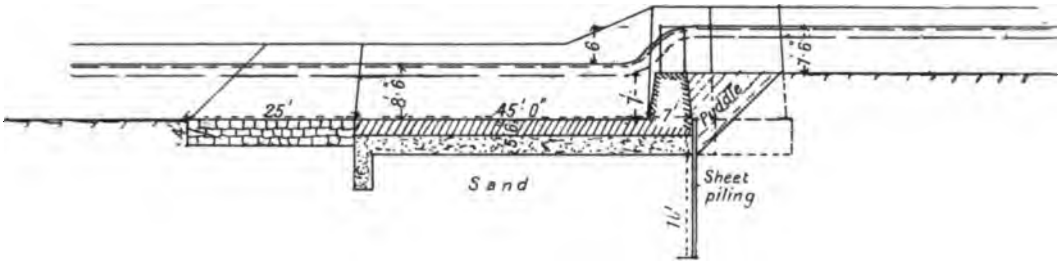


Fig. 6.—Alternative Design.

**(38)** The question often arises, in designing canal works, as to whether it is more economical to provide one deep fall or several small ones. As regards cost of masonry alone, ignoring that of the earthwork in the cutting, the advantage does not lie decidedly either way; but the wider the fall the greater will be the advantage, if any, of the deeper fall, the reason being that the cost of the flank defences bears a greater proportion to the whole, in a narrow, than in a wide work. In a long weir the main expense is in the weir wall and floor. So, ignoring the flank defences, the sectional areas of the weir wall and floor alone for different depths of fall are given below. In every case the value of *d* or depth of film is taken as 5 feet; $H_1$, therefore, will equal $H + 5$ in every case:—

| | Heights of Weir above Floor. | | | | | |
|---|---|---|---|---|---|---|
| | Feet.<br>5 | Feet.<br>7½ | Feet.<br>10 | Feet.<br>12½ | Feet.<br>15 | Feet.<br>20 |
| Sectional areas— | | | | | | |
| Weir . . | 19 | 45 | 53 | 72 | 101 | 155 |
| Floor . . | 80 | 112 | 150 | 175 | 216 | 300 |
| Totals, sup. ft. | 99 | 157 | 203 | 247 | 317 | 455 |

| | Sect. area in sup. ft. |
|---|---|
| Two 5 feet deep falls . . | 198 |
| One 10 „ „ „ . . | 203 |
| Three 5 „ „ „ . . | 297 |
| One 15 „ „ „ . . | 317 |
| Four 5 „ „ „ . . | 396 |
| One 20 „ „ „ . . | 455 |

| | Sect. area in sup. ft. |
|---|---|
| Two 7½ feet falls . . . . | 314 |
| One 15 „ „ . . . . | 317 |
| Two 10 „ „ . . . . | 406 |
| One 20 „ „ . . . . | 455 |

From the above it will be seen that the difference in quantities of weir plus floor is much the same in the case of several small falls and one larger one, except in the case of the 20 feet deep fall.

The comparative areas of the flank walls will throw the balance in favour of the smaller falls. In a very wide weir, with adoption of the wingless type, the difference either way will be but slight.

In estimating the sectional areas of the weirs the base width is taken as that of the "Hybrid" section, viz., $\frac{H_1}{\rho}$, not $\frac{H_1}{\sqrt{\rho}}$.

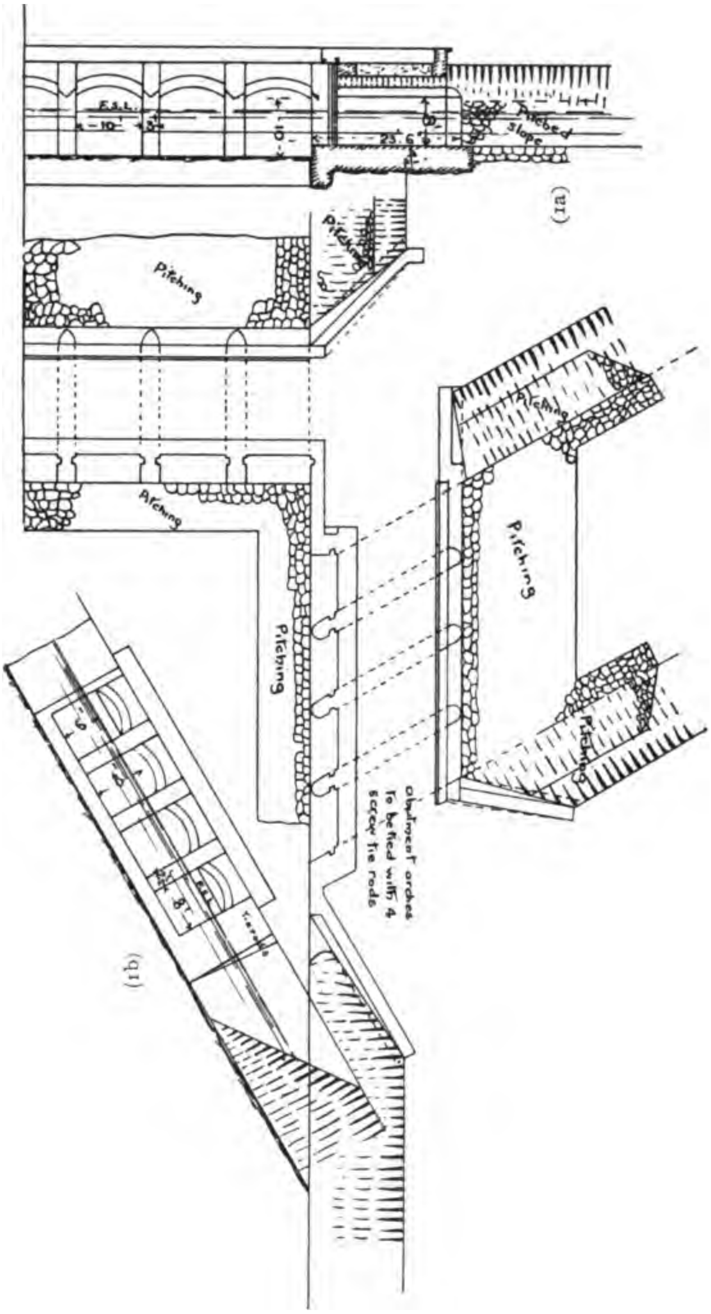
# CHAPTER X.

## CANAL REGULATION BRIDGES AND ESCAPE HEADS.

(1) In places on a canal where an Escape or a Branch takes off, a regulating bridge across both works is generally necessary. Distributaries very frequently take off above falls, and the two works are often combined. No object, however, is gained by placing the Headwork of the branch directly at the fall and so amalgamating them into one. A better arrangement is to make the branch take off separately, higher up the canal. The regulation of falls by means of a superimposed bridge with shutters or baulks which drop on the weir, is hardly practicable where notch falls are concerned, and the system is generally to be deprecated as interfering with the regimen of the canal. Skew heads for branches have a decided advantage, or, as the question of silt deposit is not so pressing as in the case of Head Regulators on a river bank, the Branch could leave the main canal at an angle, the head being recessed so as to admit of the spans being square to the axis of the branch. This arrangement, though less expensive, is not so good as a skew head, with the piers run forward to the canal side slope.

(2) The principles governing the design of canal Regulation Bridges are not quite identical with those which affect that of canal Head Regulators. In the latter, the object of the design is to enable the head work, if so required, either to completely cut off all ingress of the water from the river into the canal, or else to admit only as much as may be desirable.

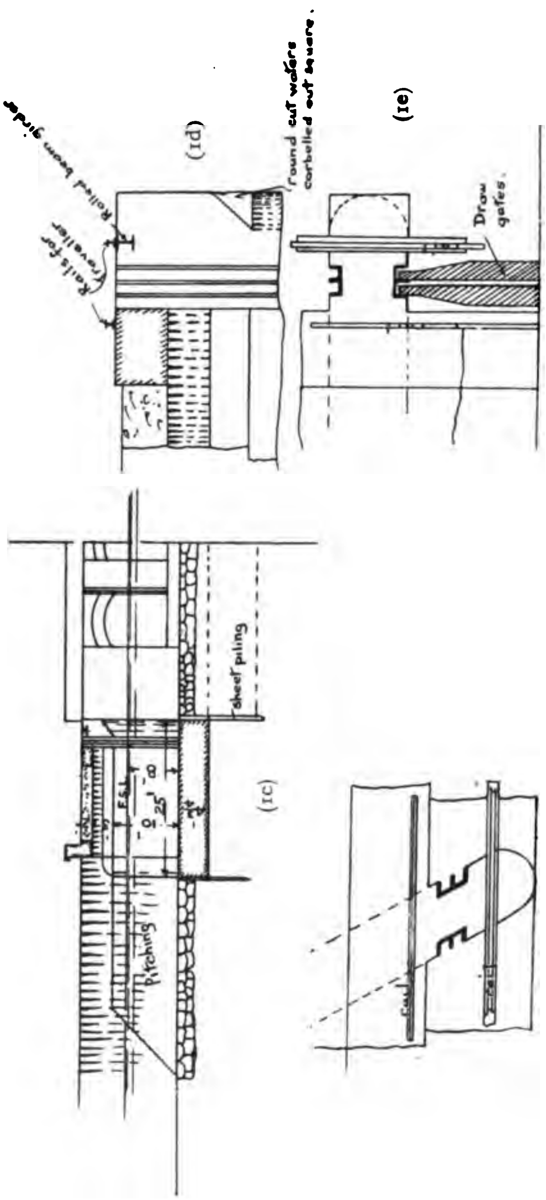
The same functions have to be performed by the canal Regulation Bridges with this difference, that the maximum head of water dealt with is moderate in depth, seldom exceeding 10 feet at the outside, whereas in a river canal Head the flood water may rise to anything up to, say, 30 feet above floor level. This fact modifies the style of design. The depressed arch, or a double series of arches at two levels, the lower carrying a breast wall, designs which are suitable for the latter works, need not be employed in the former. The springing of the arch, of which only one high level series is required, can be placed some short distance above the maximum water level. Within certain limits it is generally immaterial as regards cost, whether the arch be raised above full supply level or be depressed to that level or below it, the additional height of the piers in the one instance being nearly balanced in area by the greater depth given to the breast or flank walls. The depth of water to be regulated being small, the gates themselves can be made high enough to exclude the water without there having to be a watertight connection between the top of the gates and the face of the arch, the water being held up by the gates alone, irrespective of the spandrel walls above.



FIGS. 1, 1a, 1b.—Design for Regulation Bridge and Branch Head.

(3) A design for a Regulation Bridge and Branch Head is given in Fig. 1. In this case the water in the canal is assumed to be 8 feet deep and that in the branch 6 feet, the floor of the latter being placed 2 feet above that of the

main channel. The banks, with cutting and spoil, are assumed to be 15 feet above canal bed level, and the bridge roadway or platform is placed at this level. The spring of the arches is fixed 2 feet above F.S.L., which is the same in the branch as in the canal. The space of 7 feet between F.S.L. and the road level is useful for hanging the gates when drawn up clear of the water line. Some space between the F.S.L. and the crown of the arches is also deemed advisable to allow of row boats passing underneath. In the canal Regulation Bridge the spans are made 10 feet. Large spans with high heavy arches are quite out of place in irrigation canals, where headway need not be provided for laden barges. Most of the examples of existing canal Regulators given in Buckley's "Irrigation Works in India" are of this latter type, which it would be folly to imitate in a purely irrigation canal. The openings in the skew Branch Head, which takes off at an angle of 60° with the axis of the main canal, are made 8 feet wide, which will give an oblique span of about 9 feet. The skew arches have the same rise and thickness as in the canal Regulator.



FIGS. 1c, 1d, 1e.—Design for Regulation Bridge and Branch Head.

(4) The disposition of the wing walls in the combined work is arranged so as to allow of a suitable approach to the bridges from the canal bank road on either side, as well as from the country at the rear. The connection between the two works is formed by a short continuation of the abutment of the canal Regulator, which joins on to the end of the left skew abutment of the Branch. This is the only vertical retaining wall in

the design. The width of the roadway between parapets is 12 feet, which is the least dimension suitable for cart traffic. This makes the length of the piers and the floor in both cases about 24 feet. The head of water being only 8 feet, the proportional length of the masonry floor comes to $3H = 24$ feet, or measuring from the grooves, over $2H$, so that the width of the masonry floor is, if anything, in excess of requirements, and need not be extended beyond the down stream pier noses. The thickness of the floor is $\sqrt{H} + 1 = \sqrt{8} + 1 = 3{\cdot}8$ feet $+ \frac{S}{10} = 4{\cdot}8$ feet in the main regulator, and $\sqrt{6} + 1 + \frac{S}{10} = 2{\cdot}5 + 1 + {\cdot}8 = 4\frac{1}{4}$ feet in the branch, the subsoil being assumed to be firm clay. $H$ is the depth of full supply above floor, and $S$ the span. The addition of $\frac{S}{10}$ is for the piers.

(5) The gates will be lifted by a travelling winch, which will run on rails clear of the roadway. One rail will rest on the dwarf parapet wall provided for that purpose, and the other will be bolted on to a rolled beam some 9 inches deep built into the top of the up-stream cut-waters, which for this purpose are continued up to road level. The detail of this arrangement is shown on a larger scale in Figs. 1d and 1e. The gates, which only require a watertight connection between each other, will be in plan as shown in Fig. 1e. They will, as a matter of course, be fitted with roller attachments.

(6) The arrangement of the pitching below the two works is shown in Figs. 1, 1a and 1b. That on the canal slopes is taken 2 feet above the water line, and terminates in a slope of 45° Where the sloped pitching projects beyond that in the canal bed, a narrow strip of horizontal pitching extends on either side to form a foundation for the toe of the slope.

(7) Fig. 2 is a presentment of the Raswaniya Regulator on the canal of that name in Lower Egypt. With the exception of the down-stream wings, the alignment of which presents some novelty, the design is but a copy of the old Ganges Canal types.

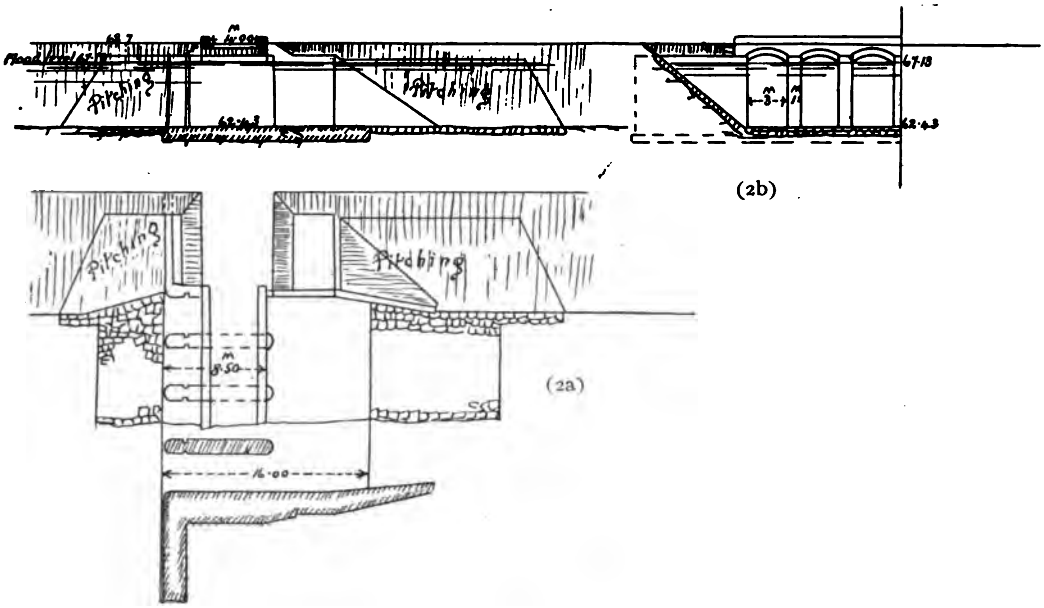
The following objections are patent to the critical eye:—

(1) Considering the depth of water, viz., over 15 feet, the spans are very narrow.

The width of openings, except in the case of Undersluice Heads, should not as a rule be much less than the depth of water, and as the regulation of this work is believed to be only partial, spans of 4 or 4½ metres, *i.e.*, 13 to 15 feet, could well have been adopted. The regulation is effected by the old primitive method of wooden sleepers or baulks dropped into grooves cut in the brick piers, which require a large and experienced gang of men to properly manipulate.

(2) In conformity with the practice in Upper India, the wing walls are vertical in section, which is the least economical form. The direct return up-stream wings are likewise an unsuitable disposition unless for a Head

Regulator taking off from a river bank, which is apparently not the case here. Reference to the elevation in Fig. 2 will demonstrate how injuriously the sudden narrowing of the water way, due to the abrupt entire suppression of the side slopes, must affect the current. A good arrangement of the water wings will, on the other hand, ensure as gradual a change as possible in the sectional area of the water way, and this can only be effected by either adopting splayed walls which gradually absorb the side slopes, or else a direct continuation of the abutments by means of a sloping crested wall, similar to what is adopted down stream in this design, but parallel to the axis of the canal, which type of water wing effects a similar gradual widening, but in a different manner.



(2b)

(2a)

Figs. 2, 2a, 2b.—Raswaniya Regulator.

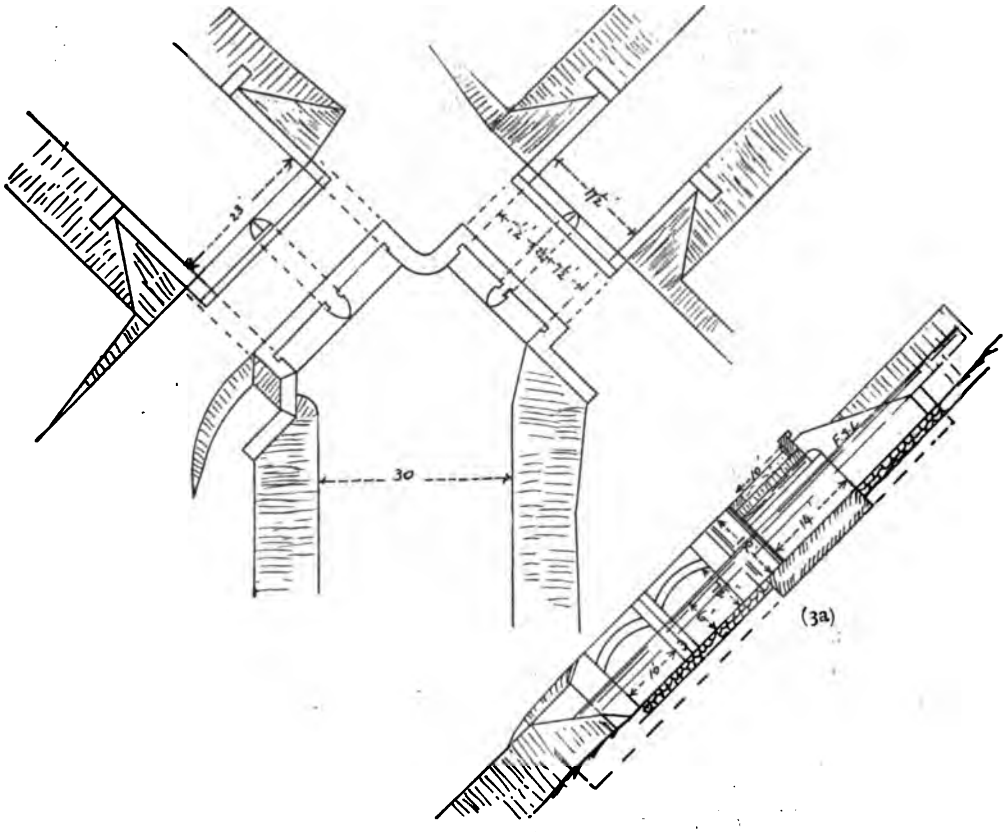
(3) The down-stream wings in this work converge on plan, the object being to force the current into the centre of the water way. The author believes that this device will defeat the very object which it is intended to secure. The contracting of the water way will tend to cause the current, on being released from the restraint of the wings, to spread out on both sides, as water flowing through an adjutage undoubtedly does.

A good point in the design is the arrangement of the pitching on the slopes, which is as it should be.

(4) The floor of the work is only 1 metre or 3¼ feet thick. These thin floors are also a relict of the old designs.

**(8)** Were the work subjected to the full head of 5 metres, the floor would be dangerously weak. It may be urged that the work will never be called upon to withstand the greatest possible head, and was designed accordingly.

However, there is no knowing when, from the occurrence of a breach in the bank or some accident below the work, it will not be found necessary to completely close a Regulator, which for this reason should invariably be designed to carry the full head of water. It is true that the Ibramiyah Head work is not so constructed. The floor of this work is greatly deficient in sectional area for this purpose, and it would have been a much better plan to have extended the floor once for all. The superstructure in both these cases is equal to whatever stress can be brought upon it: hence it seems culpable folly, by saving money on the floor to thus reduce the efficiency of the whole work. Sir William Willcocks, in his work, "Egyptian Irrigation," admits the difficulty experienced in works originally designed for partial regulation being now required, by the altered circumstances of the canal water *régime*, to deal with a greater head than they are capable of controlling.



FIGS. 3, 3a.—Design for Bifurcation.

The floor should be of a thickness of $\sqrt{H} + 1$, or 3·8 + 1, *i.e.*, 4·8 feet; add to this $\frac{S}{10}$ or 1 foot, we have a depth of 6 feet, *i.e.*, nearly double what it is now. The length of the floor at this depth is, if anything, in excess of requirements.

**(9)** Fig. 3 is a design of a Bifurcation. The depth of water is taken as 6 feet in the canal and also in the two branches. It does not call for much comment; the wing walls have been shown as vertical in order not to complicate the sketch unnecessarily; they should certainly be battered in face in an actual case. A design for a Regulator without bridge to pass traffic is given in the next section on Canal Escape Heads (Fig. 5).

### *Canal Escape Heads.*

**(10)** An Escape leading out of a canal within one or two miles of the Head is an indispensable adjunct wherever silt deposit threatens to block the water way. In addition to this cause, several similar works are required on a canal of any considerable length. Notwithstanding the fact that the admission of water at the Head is under complete control, still the supply on some of the lower reaches may be in excess of requirements, owing to heavy local rain, which would stop cultivators from using the canal water. For this reason escapes in suitable positions, whereby the surplus supply can be got rid of, are necessary.

**(11)** An Escape Head should, if practicable, be combined with a fall, as a less width of water way will then be requisite, diminishing thus the cost of the work and the number of bays to be worked. The design will then be a combination of a canal Regulation bridge and a fall.

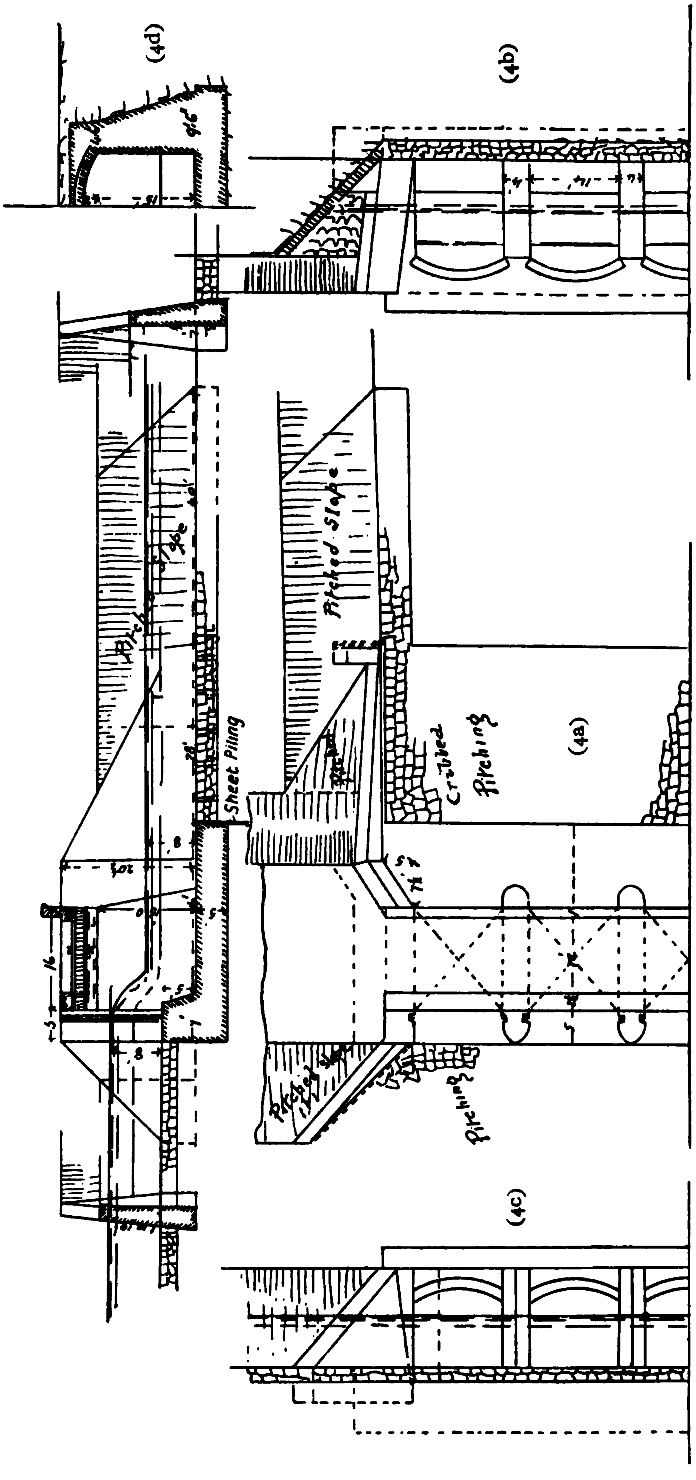
The position of an Escape Head should be as close as possible to a drainage line, so as to diminish the length of the excavation in the channel. As a rule, several falls will have to be constructed along its course, in order to overcome the difference of level between the bed of the canal and that of the river or stream into which the escape channel will tail.

Canal aqueducts crossing drainage lines can sometimes be utilised with advantage to pass off surplus water. An example of such a work is given in Fig. 8, to which reference will subsequently be made.

**(12)** Fig. 4 is a design for a canal Escape Head under the following assumed conditions:—

The bed width of the canal is 100 feet, side slopes one to one, the depth of water, full supply, is 8 feet, and mean velocity is $2\frac{1}{2}$ cubic feet per second, having a discharge of 2,160 cubic feet. Supposing the silt deposit to be of so heavy a nature that it will take a mean velocity of above 5 feet per second to scour it, then the Escape water way will have to be designed of such a width as to induce this velocity in the approach channel, when running full supply.

The site of the Escape Head is assumed at 5,000 feet distant from the canal Head, with a free overfall, the crest of which is flush with the canal bed. The slope of the canal bed is estimated to be close upon ·1 per 1,000 feet (with $n = \cdot 0225$), hence the drop in level, from the Head Sluice floor to the escape, will be ·5 feet. Increasing the velocity from $2\frac{1}{2}$ to 5 feet per second will nearly double the discharge. It is, however, presumed that the maximum supply through the Head Regulator is limited to 2,500 cubic



FIGS. 4, 4a, 4b, 4c, 4d.—Design for Canal Escape Head.

feet, *i.e.*, about 25 per cent. in excess of full canal supply. The head being only designed to pass 2,160 cubic feet through its openings with a depth of 8 feet, in order to accommodate the additional quantity there will have to be a less depth of water below the work than above it—in fact, a fall. It is presumed that this diminished depth will not exceed 7 feet, giving a fall through the regulator of about 1 foot.

**(13)** The effective length of the Escape weir will be tentatively assumed at 100 feet, equal to the canal bed width. The discharge passing over being, as before stated, limited to 2,500 cubic feet, that per foot run will be 25 cubic feet per second. We now require to find out what will be the depth of film over the crest to discharge 25 cubic feet with a velocity of approach of between 5 and 6 feet per second. According to Tables of discharges of overfalls (Table II., Chap. V.), the required depth is at once found to be 3½ feet. Thus the surface level at the escape will be 3 feet above the Head Regulator floor. We now have to ascertain what surface slope will produce a velocity of 5 feet per second at an average depth of, say, 5 feet in the channel. With this depth $R$ is estimated to be 4·6. Taking $n$, the coefficient of rugosity, as ·250 (it might well be assumed higher for silt-laden water), the coefficient $c$, suitable for a value of $R = 4\cdot6$ (and among high values of $S$ as ·7 to 1·0 per 1,000) is ·78. The mean velocity ($V$) of the channel $= 100c\sqrt{RS}$. In this case $V = 5$ and $c = \cdot78$, whence $100\sqrt{RS}$ (with $R = 4\cdot6$) should $= \frac{5}{\cdot78} = 6\cdot4$. By reference to Table VII., Part II., Jackson's "Hyd. Manual," we can find out what value of $S$ per 1,000 will correspond to one of $100\sqrt{RS}$ of 6·4. This is found to be ·85 per 1,000. Thus in a length of 5,000 feet the surface fall will be 4¼ feet; this, added to the 3 feet surface level at the escape head, will make the depth of water at the Head Regulator 7¼ feet to produce a velocity of about 5 feet per second midway.

This is near enough for our purposes. When the canal is clear of silt the mean velocity of the current will be least at the Head of the canal and greatest at the Escape Head. The velocities will vary as $\frac{Q}{A} : \frac{Q}{a}$, *i.e.*, to $\frac{2{,}500}{750} = 3\cdot33$ feet per second at the canal Head and $\frac{2{,}500}{310} = 8$ feet per second at the Escape Head. This will somewhat modify the value of $d$, which was adopted on the hypothesis of a lower velocity of approach, about 5½ feet, but not to any very great extent, probably bringing it ·2 feet lower, and will reduce the 7¼ feet at the head to 7 feet. The bed and sides of the canal will require pitching all the way, particularly near the escape head, where large blocks carefully set will have to be used to withstand the great velocity of the current.

These conditions of velocity and discharge will, however, only apply when the canal is clear of silt, that is, at the very close of the scouring operation. At the commencement, the depth, particularly near the head, will be greatly reduced owing to the silt deposit. Thus the bed slope will at first approximate to the surface slope and the current will have a fairly uniform depth of from 4 feet to 5 feet. Also the velocity throughout will be about 5 feet per second except close to the escape fall, while the amount of water passed through will be the same as at the close of the operation. The construction of a powerful scouring escape near a canal Head, if judiciously worked and with an ample supply to draw from, must effect great economy in silt clearance; as not only does it keep the head reach clear, but the periodical removal of deposit at this place will altogether prevent or greatly ameliorate the deposit of deleterious sand in the canal bed generally, and consequently effect economy in annual maintenance.

(**14**) The design of the Escape Head in Fig. 4 consists of a bridged overfall weir, the length being divided into seven bays of 14 feet. As the bridge will be used for traffic, being on the canal bank road, the width will be 12 feet between parapets. The arrangement for the gates will be similar to that in the example given in Fig. 1, the outer rail for the traveller being supported by a rolled girder built into the top of the front cut-waters. The depth of film ($d$) is taken as 8 feet (that of full supply in the canal), for purposes of sectional calculation.

The disposition of the wings down stream (Fig. 4a) is of the dog-legged type, the up-stream right wing is of the splayed sloping crested type, to allow easy access of entry for the water in the channel; with regard to the left wing down stream (*vide* Fig. 5), the junction with the Regulator is formed by a continuation of the abutment on a reduced section.

The dimensions of all the parts are in exact accordance with the formulas and rules adopted by the author as detailed below:—

Rise of arch . . . 2 feet $= \frac{S}{7}$

Thickness of arch . . 1½ feet $= \cdot 4\sqrt{r} = \cdot 4\ \sqrt{12} = 1 \cdot 4$

Thickness of abutment at springing $= \frac{r}{5} + \frac{V}{10} + 2 = \frac{12}{5} + \frac{2}{10} + 2$ $= 4 \cdot 6$ feet.

Thickness of floor $= \sqrt{H_1} + 1 = \sqrt{13} + 1 = 4 \cdot 6$ (made 5 feet).

Length of floor beyond weir wall $= 2\,(H + d) = 2 \times 13 = 26$ feet.

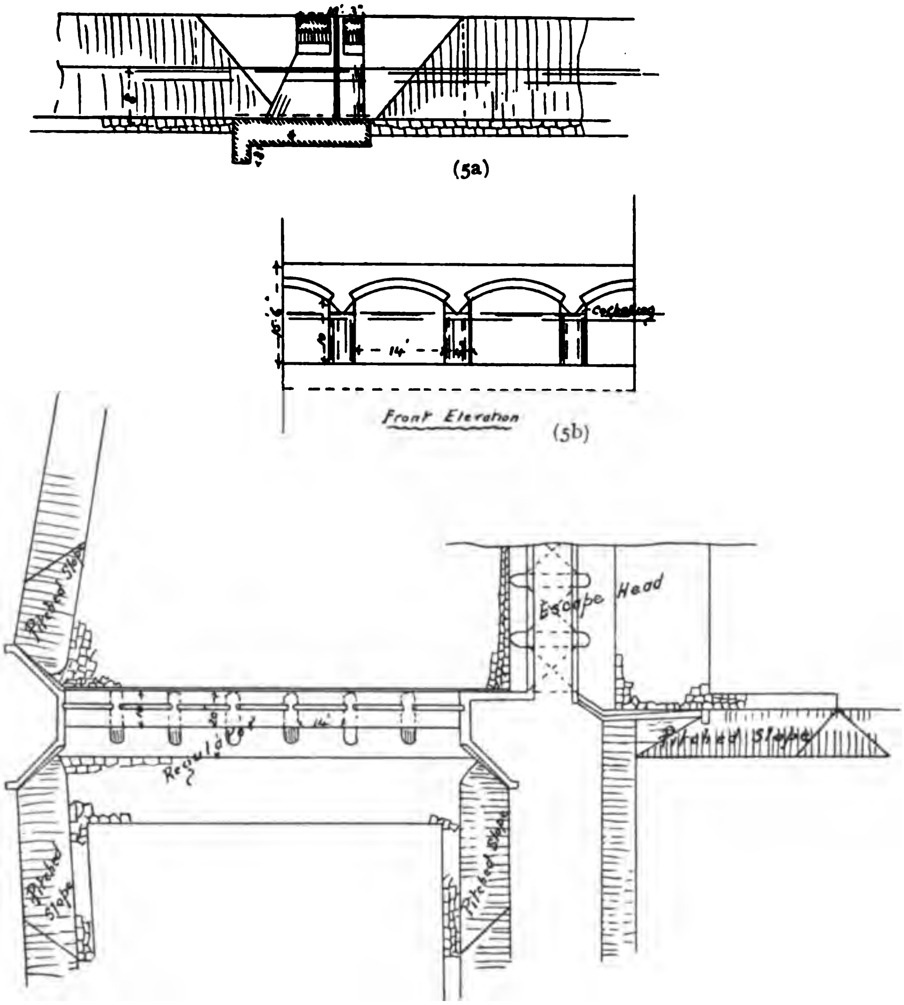
Thickness of piers $= \cdot 3S = \cdot 3 \times 14 = 4 \cdot 2$ (taken as 4 feet).

The thickness of the base of the retaining walls is in accordance with the formula $\cdot 4H - 1$ for stone walls with a face batter of one in ten. The up-stream wing has this proportional thickness at the canal bed level; below ground the back is carried down vertical, the face continuing to batter to the foundation.

(**15**) Fig. 5 is the design for the canal Regulation Bridge just below the Escape Head. It is not supposed to be required to cross traffic, but only for regulation purposes. As will be seen in the transverse section Fig. 5a, the cut-waters are corbelled out square above F. S. L., to carry the outer tier of arches which is provided, similar to the arrangements common to partial Regulators, as the Assiût works, and Undersluices generally.

The spans adopted are the same as in the Escape Head, viz., 14 feet, of which there are seven, giving a combined width of water way of 98 feet, *i.e.*, closely approximating to the bed width of the canal, which is quite sufficient for bridges and cross canal works. The canal bed is shown widened out on one side only.

A skew Regulator would really be more in place in combination with the Escape Head, but would cost more.

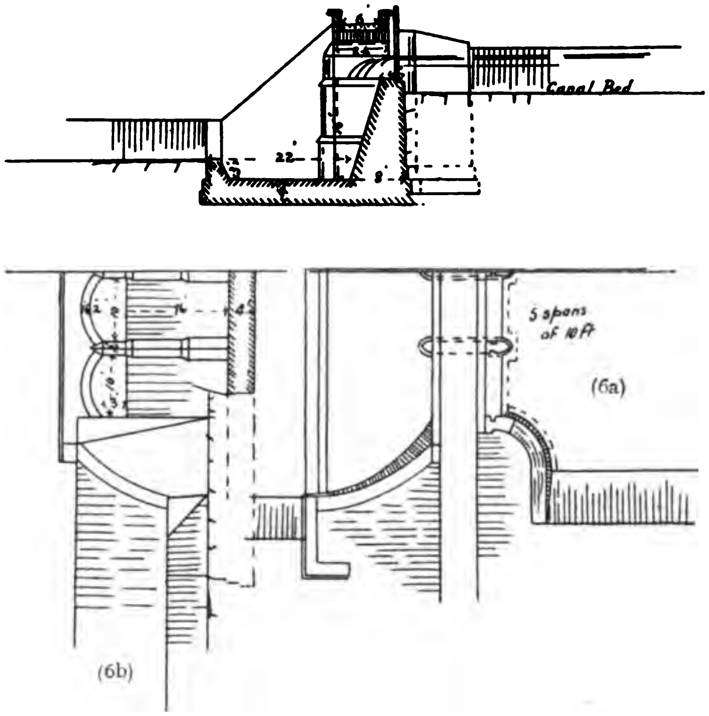

(5a)

(5b)

FIGS. 5, 5a, 5b.—Design for Regulator below Escape Head.

**(16)** We will now proceed to give an example of an existing Escape Head with fall which is on the Godaveri Eastern Canal.

Figs. 6, 6a and 6b are drawings of this work. The depth of the fall is exceptionally great, being 16 feet above floor of fall and 13 feet above the level of the escape channel below. The latter being 3 feet higher than the floor, a water cushion 3 feet deep is formed by means of a raised curtain wall. The weir is divided by piers into five spans of 10 feet. The piers are partly built over the weir, and the roadway is only 6 feet wide between parapets. From this it is evident that the bridge is not required for traffic, but is merely a foot Regulation bridge. This work seems on the whole to be well planned, although the curved wings are not commendable. Straight battered wings would answer the purpose just as well and be easier to build, besides which, the railway under-bridge style of curved wings adopted down stream, which is so common in all Madras works, is objectionable; their curved shape tending to encourage a rotary movement in the water below the weir, termed "pooling," which is very destructive in its action.



FIGS. 6, 6a, 6b.—Escape and Fall on the Godaveri Eastern Canal.

The piers are economically designed, the thickness is ·3*S* at the base and they taper by insets to 2 feet or ·2*S* at the springing of the arch of the bridge. Piers in a fall are subjected to very slight water pressure, end on, and receive great support from the weir wall into which they are built. They are not likely ever to be subjected to lateral pressure, as in the case of a regulating Head or bridge, consequently the thickness can safely be diminished to that suitable for an ordinary bridge. The depth of water liable to pass over the weir is probably not more than 3 feet or 4 feet, judging from the height of the banks of the approach channel. The closure is effected by wooden gates raised by screw gear. A small movable winch on wheels, which could be taken away and housed when not required, would probably form a better lifting apparatus; or else a row of simple wooden windlasses fixed on the cut-waters. Screw gear is only suitable for reservoir sluice gates in deep water, which are moved 2 feet or 3 feet up and down at the outside, but in the Madras Irrigation Department it is applied to every kind of water gate from Undersluices to waste weirs.

(**17**) The floor of this work is only 4 feet in thickness. This is decidedly insufficient; by the rule $\sqrt{H_1} + 1$ the thickness should be $\sqrt{20} \times 1 = 4{\cdot}5 + 1 = 5\frac{1}{2}$ feet.

The length is also deficient: it should be $2H$ at least, or 32 feet measuring from the toe of the weir wall; it is actually only 22 feet. The water cushion of 3 feet, which necessitates deepening the foundation of the whole of the work, is considered to be quite unnecessary. A floor of rubble stone or brick on edge or end, set in Portland cement mortar with a good solid foundation of concrete below, will not require any water cushion beyond what is naturally provided by the filling up of the escape channel.

The weir wall is 4 feet thick at top and 8 feet at the bottom. Being evidently of the "hybrid" type, its base thickness should be $\frac{H_1}{\rho}$, or taking $\rho$ as $2{\cdot}1$, the required thickness comes to $\frac{20}{2{\cdot}1} = 9\frac{1}{2}$ feet. The 3 feet depth of water cushion in front of the weir wall is not of any material assistance, but it adds to the depth of the weir wall, and the latter's section should have been increased accordingly.

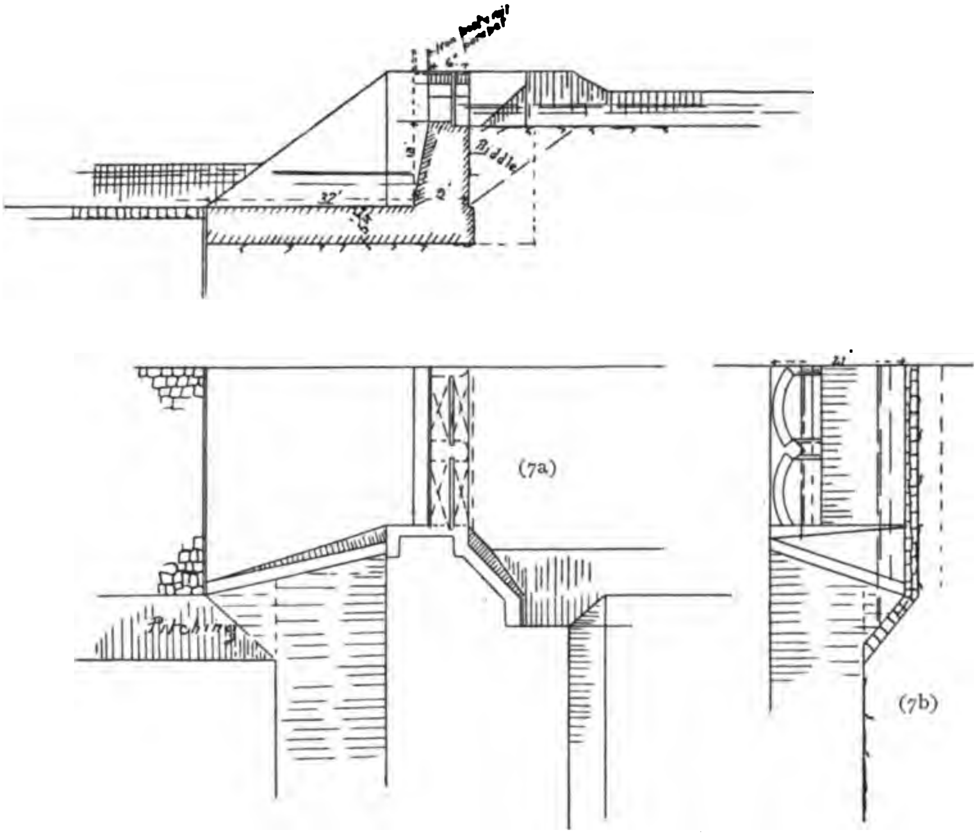
(**18**) Figs. 7, 7a and 7b are drawings of a revised design for the same work. The main alterations made are as follows: The floor is raised to the level of bed of outlet channel and its thickness increased to $\sqrt{H_1} + 1$ or $\sqrt{17} + 1 = 5\frac{1}{4}$ nearly. The length of the floor is also increased to 32 feet, or $2H_1$ nearly. A row of sheet piling is inserted for curtain protection at the termination of the masonry floor.

As only a 6 feet wide crossing is required, the crest width of the weir is increased from 4 feet to 6 feet, and the piers built on it. The archway is split into two parts to allow for the grooves and the passage of the gates. An iron post-and-rail fence is supplied on the down-stream side of the platform. In case this width is deemed insufficient, that of the platform can be increased to 8 feet as shown by the vertical dotted line in Fig. 7, the top thickness of the weir being reduced to 4 feet, as shown by the inclined dotted line. The piers will still not project over the floor, where they would reduce the area of the pool, and so increase the velocity of departure, which is to be avoided if possible.

If a still wider bridge is required, it could be formed 12 or 13 feet in width over the 6 feet piers by using longitudinal rolled girders (in place of masonry arches) with cross joists acting as cantilevers over each pier.

This construction would be much cheaper than building piers on top of the floor, and preferable as well, as it does not contract the waterway below the weir in any way. In Fig. 7b the up-stream cut-waters of the dwarf piers are shown rounded below and corbelled out square above, to carry the arch. This arrangement could be made easier by projecting the noses of the cut-waters 12 inches beyond the inner side of weir crest, the projection being corbelled out. Down stream, no cut-waters are provided; they could easily be inserted by projecting them beyond the crest and building up from the base of the weir.

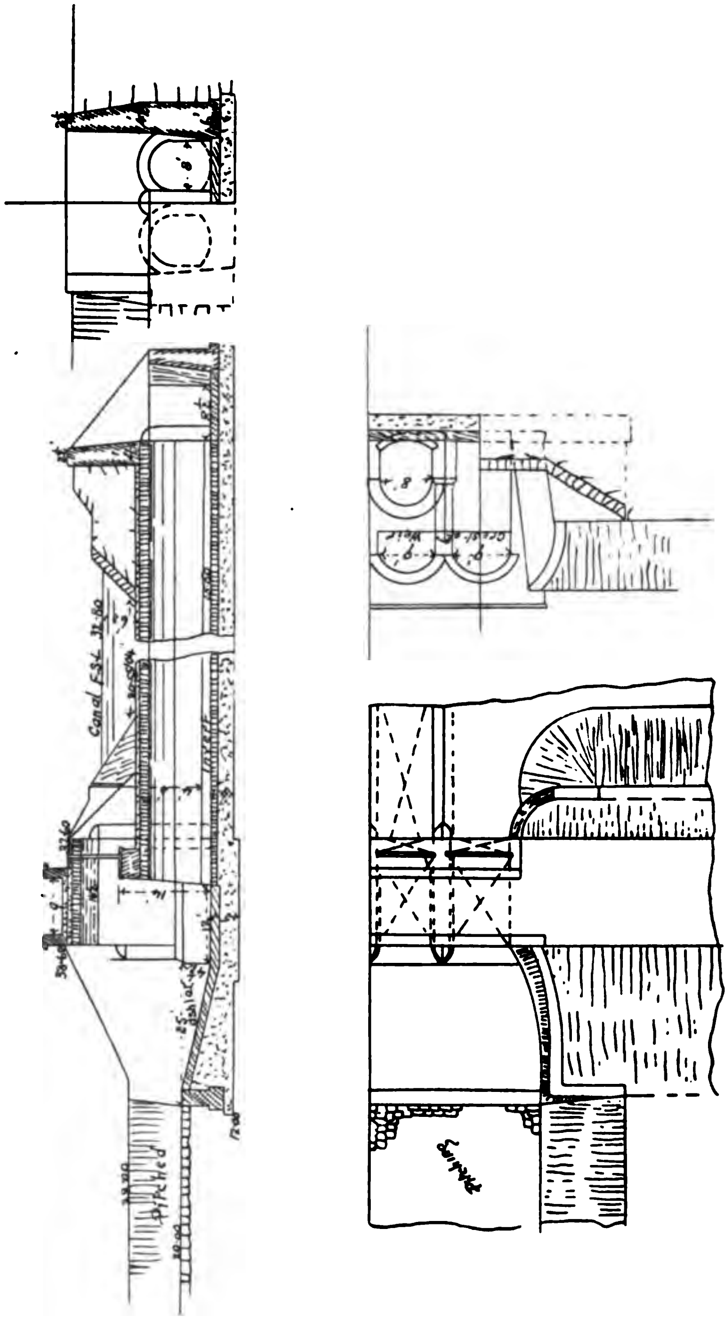
The wings are similarly disposed as in Fig. 6, but altered on plan to straight splayed walls. The revised design will probably have less material than the original in spite of the considerable increase in some of the dimensions, as the depth of the foundations will be less all round and the piers beyond the weir crest, which are 25 feet deep, will be saved entirely.



FIGS. 7, 7a, 7b.—Alternative Design.

(19) This weir evidently does not lie on the side of the canal, but is situated on an escape channel. There is probably no Escape Head proper, the escape channel leading out at bed level without a Regulation Bridge, the regulation being effected at the weir, which is some way off. The objection to this arrangement is that the approach channel to the weir will always be full of water, and if not very short will cause wastage, unless it could be utilised as a supply channel to one of two distributaries as well as an escape channel.

(**20**) It has already been noted that escapes for surplus water can easily be arranged for at aqueducts or drainage crossings, the escape or surplus weir being combined with the latter work. An excellent example of such combination is given (Fig. 8) of a work on the Connamur Canal, Madras.



(8b)

(8a)

FIGS. 8, 8a, 8b.—Combined Waste Weir and Drainage Syphon, Connamur Canal.

The canal here crosses a drainage line, which is taken underneath it by a syphon or syphon aqueduct. This consists of two culverts 8 feet wide and about the same height. The waste weir and regulating and traffic bridges are built at the end of these two culverts, half the weir being simply a dwarf wall built over the end of the barrels, the other half lying beyond them. The three centre piers are spaced 9 feet apart and are built in part directly over the central pier and the two abutments of the syphon. The weir consists, however, of five spans of 9 feet, two of which lie outside the syphon altogether, and here the weir wall is of the full depth of 14 feet. Both abutments are therefore quite clear of the syphon.

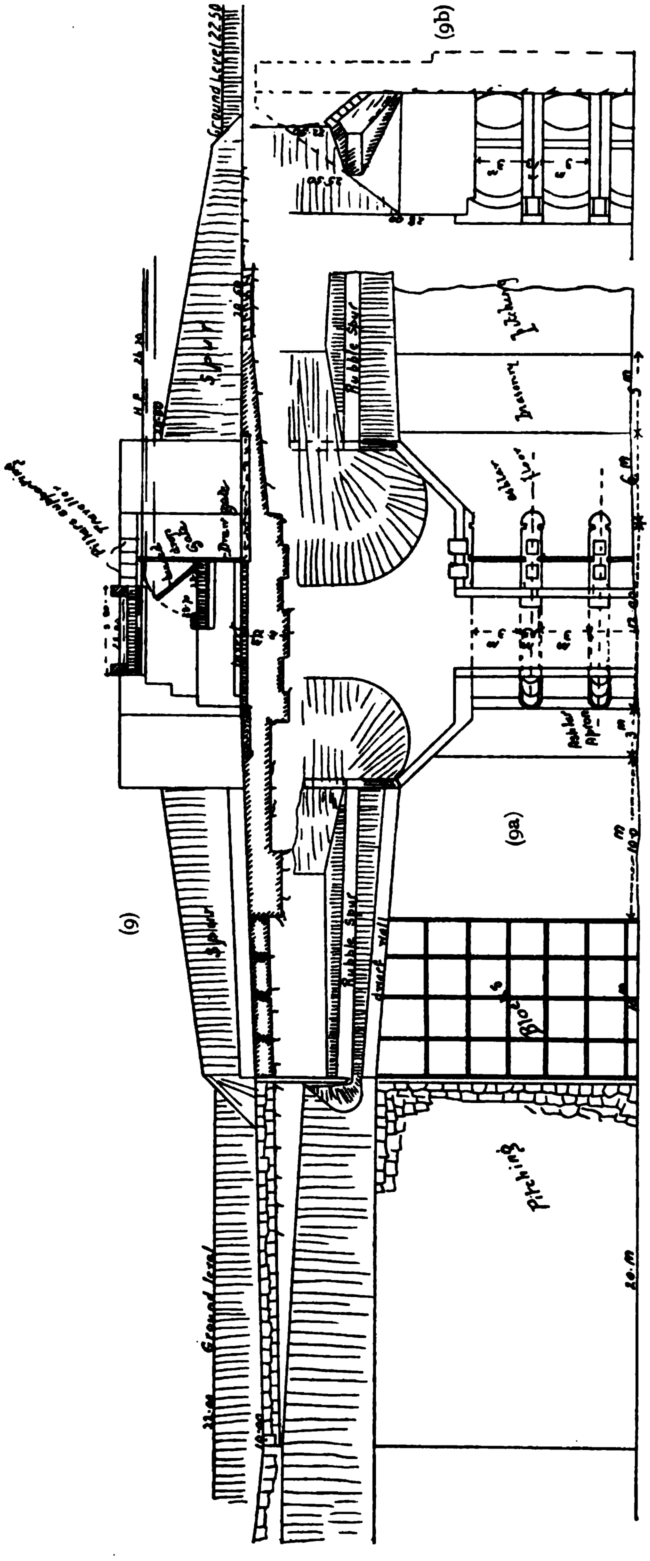
The question now arises, Is this the best arrangement possible? Would it not be preferable to lengthen the culvert and place the whole length of the three bridge piers on top of the syphon, thus saving a depth of 11 feet off two-thirds of the three piers of the regulating bridge, the abutments remaining the same depth as before, but recessed behind the face of the syphon? A great deal of space, which means increase in the length of the syphon, seems to be occupied by the canal slopes on either side. That on the left could well be reduced from 1½ to 1, as any soil, even pure sand, will stand at 45° if pitched. This 10 feet is just equal to the lengthening required to bring the bridge entirely over the syphon. Another 10 feet in length could very well be saved on the other side; this would be sheer gain. By thus recessing the bridge, the floor beyond the weir, which is unnecessarily wide, could be reduced, as the difference of level between canal full supply and the bed of the drain is 13 feet, and a length of $2H_1$ or 26 feet would be sufficient. The actual length of the floor is 34 feet, hence 8 feet can well come off here. If the design were thus modified, the saving would be 10 feet length of syphon, 8 feet of flooring 5 feet thick, and the whole of the piers below weir crest level, a by no means inconsiderable amount, equivalent to 12 or 15 per cent. of the whole.

The general arrangements of this design are otherwise decidedly good, and it forms a very instructive example.

(21) An example of a very large escape, which belongs more properly to reservoirs than to canal works, is the Koshesha Escape in Lower Egypt.

Figs. 9 are plans of this immense work, which consists of sixty spans of 3 metres, or 10 feet, and it upholds 6·2 metres, or say 22 feet, of water, although it is stated in "Egyptian Irrigation" to be subjected to a head of 4 metres only. This is too vague to enable a calculation to be made of the horizontal thrust of the water on the work, though it is sufficient for the vertical pressure on which the design of the floor may be made to depend. The soil is good hard clay.

The main peculiarity of the design is the division of the waterway into two parts by arches thrown between the piers, which thus form an intermediate platform. The upper and larger portion of the vent above this platform is closed by iron gates, hinged at the bottom, which, when released, fall automatically on to the platform to which they are hinged. They cannot be raised when the level of the water is much above that of the platform, but



Figs. 9, 9a, 9b.—Koshesha Escape (Egypt).

are intended, when the reservoir has to be emptied, to remain down till the water has run off, no regulation by them being possible. The vents below the platform are closed by ordinary draw gates working in grooves; these only come into use when the water above the level of the platform has been drained off. A 12 feet roadway between parapets is provided, so that evidently provision has to be made for crossing traffic. This and the considerable projection of the piers in front of the gates, apparently to allow a wide space between an outer set of emergency grooves and the face of the barrier gates, causes the piers to be of exceptional length, which is not required for statical resistance. This work is evidently erected in the middle of a long embankment crossing flat country, similar to a sluice in a tank embankment; it is not situated in a depression or marked natural watercourse.

The arrangement of longitudinal rubble spurs above and below the escape, to guide the current in a direction normal to the axis of the work, is admirable, and so is the disposition of the abutment wings.

(22) In many other points the design of this work is, however, open to criticism.

Taking the floor first, the depth under the piers, which depth runs right through, is considerable, being 2·75 metres, or 9 feet. Considering that the soil is hard clay, and the head of water nothing very remarkable, being 7 metres, or 22 feet, when the bed below is dry, but stated not to exceed 4 metres, or 13 feet, this mass of rubble masonry appears excessive.

The Assiût works, built not on hard clay but Nile mud, have a depth of foundation of 3 metres, or 10 feet, with not far short of the same head, viz., 3·25 metres in the Ibramiya Head.

The foundations appear to be designed very similar in style to the Nile Barrage—witness the great length up stream; whereas the conditions here in hard solid ground are quite different. There appears to be no reason why sheet piling should not enclose the foundation on either side, as was adopted in the Assiût regulators, and this widening of the floor up stream would then be quite unnecessary, and the material thus saved would obviously be better utilised below the work.

With regard to the superstructure. The provision of spare grooves and the consequent extension of the piers up stream, a construction always to be avoided if possible, appears unnecessary precaution, particularly in view of the fact that the work will only be used once a year, and will doubtless remain clear of water for several months, when any repairs to gates, etc., could be carried out. The floor should, however, be so constructed that it will never require repairs; this can always be ensured by using good cement masonry of sufficient depth. The ashlar slabs used in this and many similar works the writer considers as highly objectionable, being very liable to be ripped up, and being besides very expensive. Good hard rubble on edge with thick joints of cement will stand erosive action much better than any more expensive material.

(23) The division of the spans horizontally, though doubtless an excellent arrangement in itself, is carried out by the insertion of a great thick vaulted masonry platform, which takes up a great deal of the waterway, necessitating more spans to carry off the discharge, than would otherwise be requisite. This obstruction is not short of 5 feet in depth. Here is a case in which iron could be used with great advantage in place of masonry. The falling gate, hinged at the bottom, necessitates a long platform to receive it when down. If a gate hinged at the centre of pressure, *i.e.*, at one-third of the height, or thereabouts, be substituted, it could be manipulated with great ease, even under full pressure, that on the upper and lower leaves being balanced. It could also be arranged to fall automatically when the impounded water

rises above a fixed level. A balance gate of this description is hinged at both ends on a steel axle, which is either built into the piers or else is received into a movable plummer block, which, provided with anti-friction rollers, can slide up and down an iron groove. The gate after being pulled into a horizontal position can then be raised by chains attached to the blocks in the grooves when offering the least resistance to the water pressure. This, in certain cases, is a more suitable arrangement than that adopted in Fig. 10. The total pressure on each axle will be $\frac{wlh^2}{4}$ or in the case shown in Fig. 10c, $l = 16$ feet, $h = 13$ feet and $w = \frac{1}{36}$ ton, whence the expression becomes $\frac{16 \times 169}{4 \times 36} = 19$ tons, or taking 6 tons as safe shearing stress for steel, the sectional area of the axle will be $\frac{19}{6} = 3\frac{1}{6}$ square inches nearly, requiring to be 2 inches in diameter.

(**24**) Considering the depth of water (viz., 22 feet), the spans of 10 feet width are decidedly too narrow. They could be well increased to 5 metres, or 16½ feet, the pier being 2 metres wide, or $\cdot 4S$. A revised design comprising these alterations is given in Figs. 10.

The spans are increased to 5 metres, and in place of the heavy masonry platform, a *Z*-shaped rolled beam is substituted, on the upper flange of which the base of the swing gate abuts, while the lower draw gate presses on its lower flange. The upper swing gate is 4 metres or 13 feet, the lower draw gates 2½ metres or 8½ feet deep. The latter can be raised 3½ feet while the upper is down. The depth of the free waterway is greatly increased by this arrangement, viz., from 5 metres to 6 metres, or from 16½ to 20 feet. The sixty spans of 3 metres give an effective waterway of $60 \times 3 \times 5 = 900$ square metres, or say 10,000 square feet. With the alterations shown in Fig. 2, each span of 5 metres has a free waterway of $5 \times 6 = 30$ square metres; the required number of spans of 5 metres will then be $\frac{900}{30} = 30$ spans, giving a length between abutments of $(30 \times 5) + (29 \times 2) - 208$ metres. The existing work has a length between abutments of $(60 \times 3) + (59 \times 1 \cdot 3) =$ 257 metres ; the revised design will thus cost 20 per cent. less, and with the further reductions in the superstructure and floor (if the latter were reduced to reasonable dimensions), the saving would be quite 25 per cent.*

(**25**) A sketch of the swing gate, an invention of the author, is given in Figs. 10 and 10c. It is shown clear of the archway, but could be fixed underneath it if so desired.

The arrangement for revolving the gate consists of two endless chains attached to projections on either side of the top of the gate, which chains work inside curved iron grooves. Each chain at the top of the piers is carried round a winch drum connected by a horizontal shaft, revolving which either way causes the gate to be lowered or raised to any extent. If the

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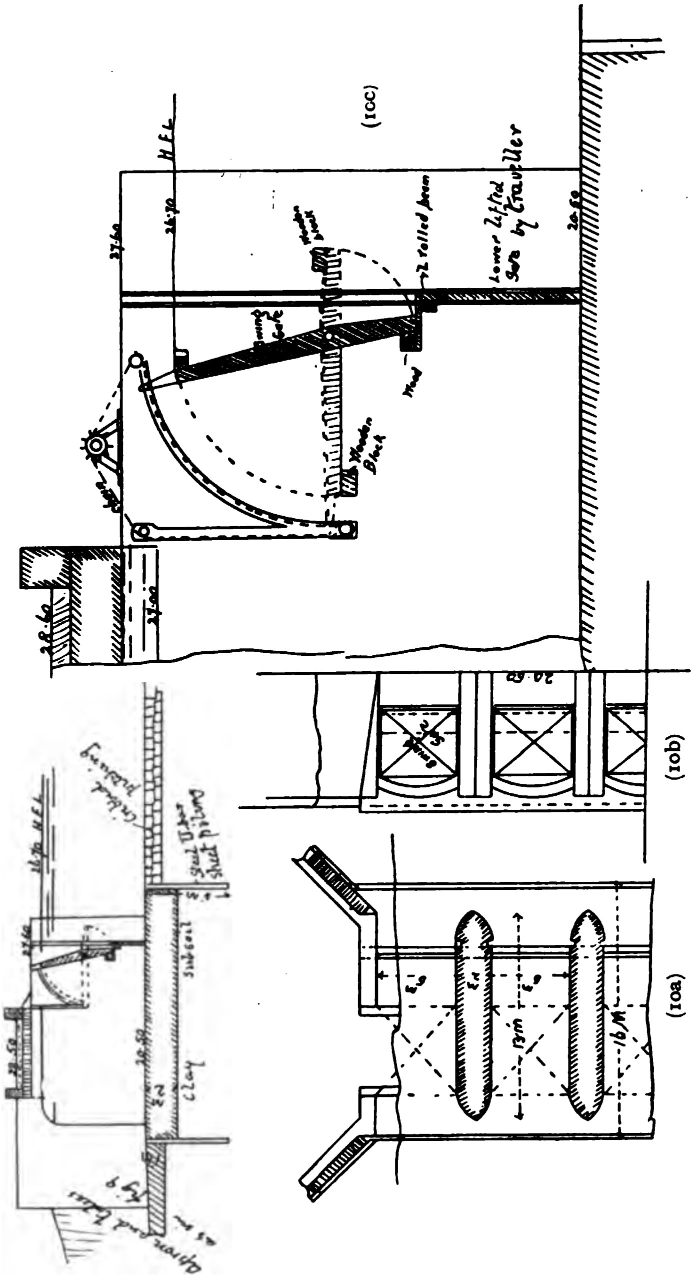
* *It has since been ascertained that the Koshesha Escape passes water in both directions, that in the reverse direction being by the lower gates alone. This fact would necessitate some modification of the alternative design in Fig. 10, viz., the lengthening of the lower half of the piers up stream by 1½ metres, and the substitution of masonry floor for the cribbed pitching beyond.*



FIGS. 10, 10a, 10b, 10c.—Alternative Design for Koshesha Escape.

gate is required to fall automatically, or at once when released, in the former case the axle can be fixed at such a height that when the water reaches a certain point the gate will revolve of its own accord, overcoming the friction of the chain and winches. If, on the other hand, the gates are required to be suddenly released with full supply on, the winch drums can be prevented from turning by a catch, which being knocked away, the gate will be free to revolve.

In the design in Figs. 10b and 10c the gate is not intended to be lifted, but when horizontal to remain in that position, with water passing above and below it. If it is deemed desirable to lift the gate, grooves can be provided as shown dotted, with blocks for containing the axles of the gate. These blocks are attached to chains lying in the vertical grooves, by which the whole gate can be lifted up by a traveller. The arrangement adopted in the plan of pivoting the gate on a fixed axle on which it swings, and leaving it *in situ,* is simpler and better, the only objection to this arrangement being the possible accumulation of floating *débris* on the end of gate. This, however, can be easily got rid of by turning the gate round a little for a time, when it will be swept away by the current.

(**26**) When an Escape is not required for scouring purposes, a Regulation Bridge across the canal is not essential. The Escape Head will then consist either of a simple Regulating bridge with its sill level with the canal bed, or raised a foot or two above that level, forming a dwarf weir wall on which the regulating gates fall; or else of a bridged fall similar to Fig. 4, only with the crest of the weir raised 2 or 3 feet above canal bed level. The length of the weir should be such as to take off the whole or a part of the full discharge as may be required.

(**27**) Distributary Heads require but little cement; their construction should be similar to that of canal branch Heads or cross canal Regulators, *i.e.*, an open waterway with or without a raised sill, closed by draw gates.

For such works double wooden gates can well be used. These can easily be fitted with small iron rollers, the lateral stanching being effected by vertical stanching strips fastened to the gate at their upper extremity only. The pressure of the water forces these against either the base or the side corners of the tables of the iron grooves. These gates can be easily manipulated by a wooden windlass fixed over each span, the barrel fitted with ratchet and pawl. The windlass can be turned by detachable wooden bars fitted into holes, or else a spoke wheel can be fixed in the centre of the drum. The system so much in vogue in Madras, of constructing canal and distributary Heads on the principle of small sluice ways situated at floor level, which are closed by a gate operated by screw gear, is much to be deprecated. This system is only suitable for outlets from tanks and reservoirs under a considerable head of water.

# CHAPTER XI.

## CANAL CROSS-DRAINAGE WORKS.

(1) THE disposal of drainage that is intercepted by a canal can be classified as the following :—

I. By lateral diversion, *i.e.*, by excavating a channel parallel to the canal, the stream can be thrown into another drainage line, for the disposal of which provision has been made.

II. By passing it underneath the canal, the canal either crossing the stream on a raised aqueduct, or, if the headway is insufficient for a clear passage, the bed of the stream is depressed below normal level, and the water passes in a tunnel underneath, rising again on the further side. This latter is termed a syphon or syphon aqueduct.

III. The drainage water can be admitted into the canal itself. This is termed an inlet.

IV. The drainage can be taken into and across the canal at the level of the bed of the latter, the inlet on one side and the exit on the opposite side. This involves one regulator across the canal and one at the farther bank across the exit of the drainage. This is termed a level crossing.

V. The drainage can be taken over the canal by an aqueduct. This is termed a super-passage, to distinguish it from an aqueduct proper.

VI. The canal can be taken under the drainage line by a depressed syphon or syphon-super-passage.

(2) In Fig. 1 we have an example of Class I., being the Thora Nala Aqueduct on the Midnapur Canal. In this the canal bed is 25 feet above that of the river, giving sufficient headway to pass the highest flood, which is 18 feet deep.

Like all masonry aqueducts, the construction mainly consists of an arched bridge with platform at canal bed level, and provided with two solid parapets, which retain the water flowing through.

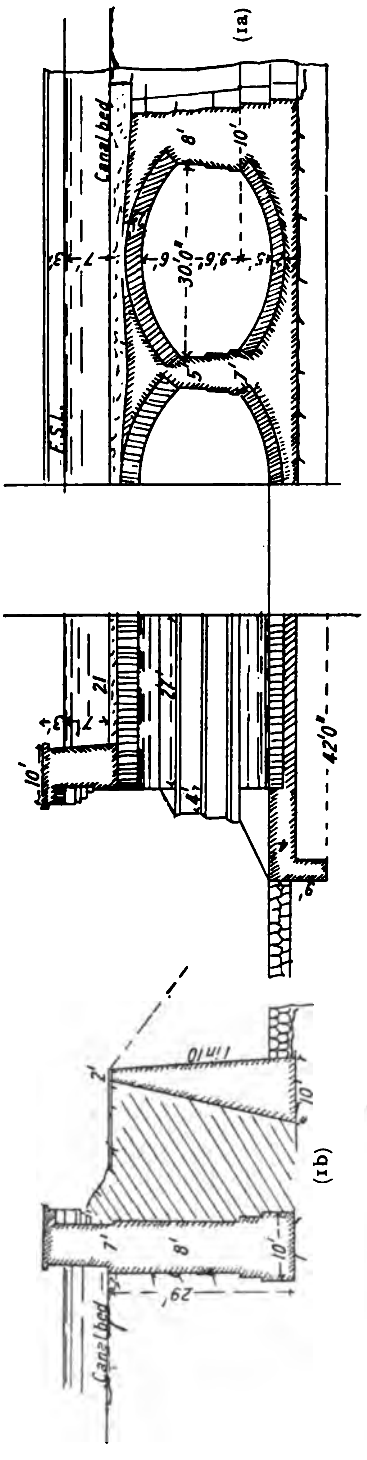
To reduce expense, the waterway of an aqueduct is made narrower than the average width of the canal in earthen banks. Owing to the smoothness of the sides, the coefficient of rugosity ($n$) is much less than that applicable to channels with earthen sides and bed, being ·013 in the former against ·025 or ·0225 in the latter. This alone greatly increases the velocity, so that a considerable reduction in section can be effected, even if the original mean velocity of the current were retained. As, however, that velocity can safely be increased to 5 feet per second, if sufficient bed slope be given, a still further reduction in the width of the water way in the aqueduct can be effected. In the example we are considering, the reduction in bed width is not shown on the general plan in Fig. 1d.

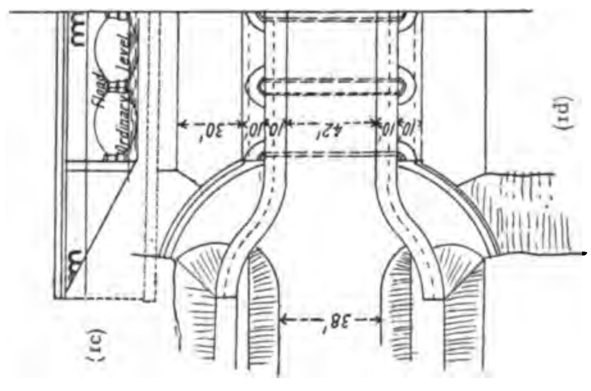
(3) The thickness of the arch throughout is 2½ feet, the radius being 20 feet. The value given to $n$ in the formula $n\sqrt{r}$, adopted for crown thickness (*vide* par. 31, Chap. IV.), is therefore ·55. The weight of water overlying the arch is $7 \times \frac{1}{36}$ tons = ·2 ton per square foot.

The live load in ordinary bridges is assumed at about 120 lbs. per square foot. This represents an equivalent weight of water of 2 feet in depth. Therefore the extra live load, which, however, is not suddenly applied and removed, as in the case of railway and road bridges, will be the weight of the depth of water carried, minus 2.

The value of the coefficient $n$ in the formula $n\sqrt{r}$ can then be approximately increased in proportion to the depth of water carried; in ordinary cases $n = {·}4$.

The following rule for deducing the increase to the value of $n$ will suit in most cases. Let $d$ = depth of water, then $n = {·}4 + {·}02\,(d - 2)$. When the depth of water is 2 feet, $n$ will remain ·4, with 5 feet carried $n = {·}46$, with 6 feet $n = {·}48$, with 7 feet $n = {·}50$, with 10 feet $n = {·}56$. This is for large spans of over 25 feet. For smaller spans $n$ should be taken as ·5, no matter what the depth of water is.





Figs. 1, 1a, 1b, 1c, 1d.—Thora Nala Aqueduct.

In the example of the Thora Nala Aqueduct the correct crown thickness would then be, with 7 feet water carried, $\cdot 5 \times \sqrt{r}$ or $\cdot 5 \times 4\cdot 5 = 2\frac{1}{4}$ feet. This would increase to $2\frac{1}{2}$ feet at the springing.

The parapets, as is usual in large aqueducts, are widened out to carry a roadway, as communication for cart traffic must be kept up along canal banks. The parapets here are 7 feet wide at base and 6 feet at top, corbelled out to provide a 10 feet roadway, with an iron rail fence on either side. The thickness of the parapet in this case is excessive. It can be made half, but should not exceed two-thirds, the depth of water. In the next example we shall see it is made about $\frac{2}{3}D$ in width.

(4) The piers are $\frac{S}{6}$ or $.167S$ in thickness. They widen out by offsets to 7 feet, or $\cdot 23S$ at the base.

There is no definite rule regarding the ratio of the thickness of piers proportional to the span in the case of large span bridges. It may be taken to vary from $\frac{S}{10}$ to $\frac{S}{5}$.

For heavy works of this description the proportion $\frac{S}{6}$, as in this case, would not be excessive. In the Kali Nadi Aqueduct of 60 feet spans (Fig. 3) the proportion is $\frac{1}{8\frac{1}{2}}$, and in the Conneram Aqueduct (Fig. 4), with spans of 40 feet, the proportion is $\frac{1}{7\frac{1}{2}}$ nearly, or, to be exact, $\cdot 15S$. In the Budki Super-passage (Fig. 8) the proportion is $\frac{1}{5}$. All these carry about 7 feet of water, and are all built of brickwork. A safe rule to adopt would be to make the top thickness of piers of large span aqueducts for 25 feet span and over $\sqrt{S}$, below 25 feet $\cdot 2S$ (*vide* par. 12, Chap. IV.).

The piers of large span bridges should increase in thickness towards the base with the object of better distributing the pressure on the foundations, and further of inducing a uniform stress at all points in the height of the pier. A formula for effecting this is given in "Molesworth's Pocket Book," p. 89.

In the example we are now examining, the width of the pier increases by two offsets from 5 feet to 7 feet at the base. A straight or curved batter would have been a simpler and better construction.

(5) The floor is composed of inverted arches with a versed sine of 5 feet, the thickness in the centre of the span being 4 feet, and that at the spring line of the inverts 9 feet. The object of this invert is evidently to distribute the weight on the piers evenly over the somewhat shallow foundation. It is very doubtful whether the inverted arch does really act in this way; the use of an invert is more to prevent a floor from blowing upwards from water pressure underneath, and it is used with advantage for this purpose in works subjected to a head of water. In the cases of bridges, however, there is no appreciable head of water against the work, consequently inverts are, apparently, not necessary.

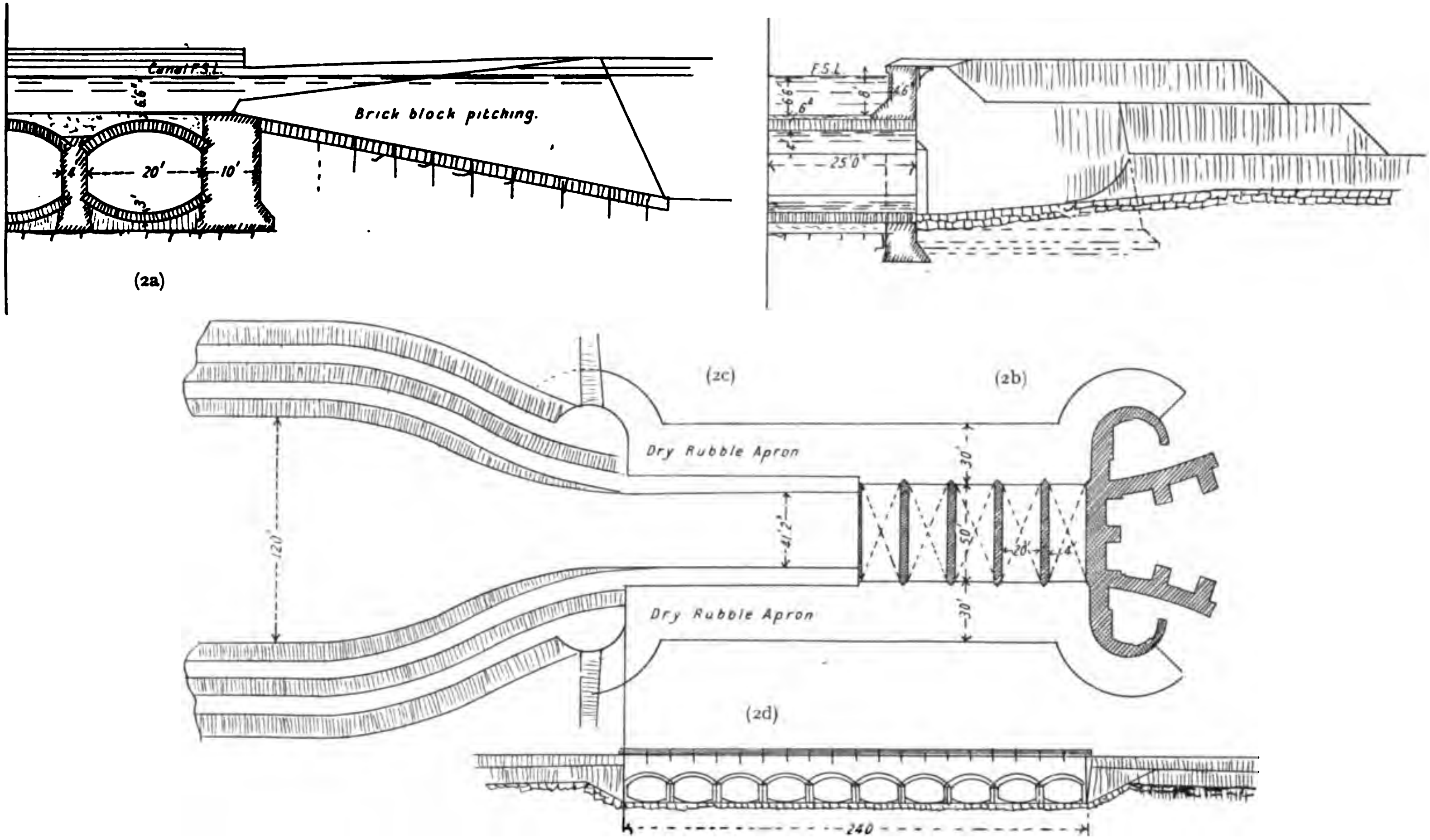
The objection to their use is the great obstruction they offer to the free passage of water, by decreasing the effective depth to the extent of nearly 3 feet. It would have been far better to have made the foundations thicker all through, say 5 or 6 feet deep, with a level floor flush with the bed of the river. The projection of the invert is eased off by a sloping apron at both sides and in front (Figs. 1 and 1d).

**(6)** The sections of the abutment and of the wing walls (Fig. 1b) are good. Figs. 1c and 1d are part elevation and plan at a smaller scale. Reference to Fig. 1d will show the disposition of the wings, always a most important point. In almost all aqueducts and super-passages double sets of wings are required, viz., two long curved land wings to form the connection between the masonry aqueduct and the earthen banks of the approach channel, and two water wings connecting the face of the abutments with the river banks on either side. The land wings form really a continuation of the parapet walls, and are of the same section at the top. Being subjected to hardly any earth pressure, they can be built with vertical sides, of the same width throughout as the top. The section in Fig. 1b is wasteful on account of the parapet being designed of unnecessary solidity, which involves the land wings in the same fault. We shall see that in other examples, by a better arrangement, an equally wide roadway can be provided without unduly thickening the parapet and wings.

**(7)** Figs. 2, 2a, 2b, 2c and 2d are different views of the Kerai aqueduct on the Sôn Canal. The construction is very similar to the last example, the main peculiarity of the design being this, that to obtain the necessary headway, the spring lines of the invert arches are made on a level with the bed of the drainage line, the crown and floor proper being depressed 3 feet below the normal bed. A flat pitched slope connects the two levels. If the flood line rises above the archway, the aqueduct will become a syphon aqueduct. The drawing in "Irrigation Works in India," from which this plate is prepared, does not, however, show the flood line. This device of depressing the floor to avoid the obstruction offered by the invert is a good one, *i.e.*, on the assumption that the invert is necessary. The author is, however, strongly of the opinion, stated with reference to the same matter in the last example, that a better design would be to increase the thickness of the floor, which is only 3 feet, to, say, 5 feet, and do away with the invert altogether. The bases of the piers are carried through the concrete to the ground—a bad arrangement, which tends to concentrate, instead of distributing the weight of the superstructure. The thickness of this mass floor is determined by the limit pressure allowable on the soil. The bearing area of the pier base increases 1 foot in width for every foot in depth of the floor (par. 9, Chap. IV.).

The invert stops dead short at the termination of the piers, the sloping continuation peculiar to the last example not being adopted.

**(8)** The parapet in this example is 4½ feet thick, or just about $\frac{2}{3}D$; it is widened inwards at junction with the arch by the addition of a triangular strip,



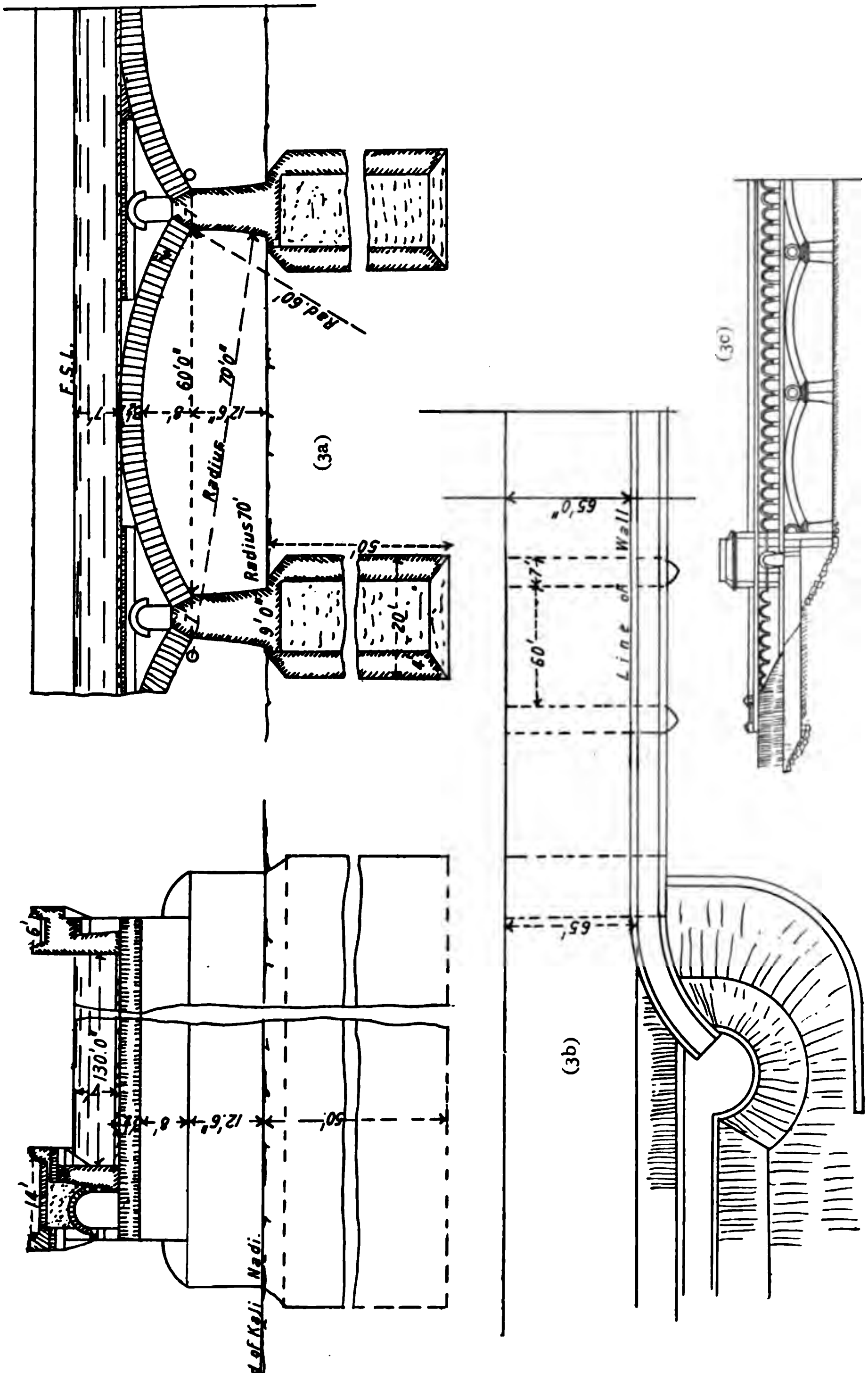
FIGS. 2, 2a, 2b, 2c, 2d.—Kerai Aqueduct, Sôn Canal thickening the actual base to 8 feet and filling up the corner. This is an excellent arrangement. The widened roadway along the top of the parapet is formed by a projecting cornice on the inner side and by iron brackets, probably planked over, with a fence on the outside. This forms a roadway 8 feet wide, giving just room for a cart to pass. The arch is 2 feet thick, *i.e.* very nearly $\cdot 54\sqrt{r}$, the radius being 14 feet.

The section of the abutment, which is 10 feet thick, or half the span, and with the buttress (*vide* Fig. 2b) is 16 feet thick in the centre, is a sheer absurdity. If made 5 feet wide at top and 10 feet at base, the dimensions would be ample. The bed of the canal is apparently not made up level with the top of the abutment, but slopes downwards towards the ground surface. This slope is pitched and likewise both the inner slopes of the canal banks. This hollow is probably left to silt up naturally.

The disposition of the wings is shown in Figs. 2b and 2c. The water wings are semicircular in plan and are very high, so that the land wings are subjected to very slight unbalanced earth pressure. The widening of the canal from 40 feet to 120 feet is shown in Fig. 2c. These dispositions must be pronounced good. The semicircular water wings form a very efficient protection against the scour of the river in flood.

**(9)** Fig. 3 is a representation of the Nadrai Aqueduct over the Kali Nadi, on the Lower Ganges Canal. This is one of the largest single works erected in Upper India, consisting of fifteen spans of 60 feet and a width between parapets of 130 feet. This is the second work erected at this site within a short period. The first aqueduct was designed to pass a maximum discharge of 5,000 cubic feet per second in the stream, and was completed and in working order when, after a period of exceptionally heavy rainfall, the Kali Nadi, which is ordinarily a simple depression, the banks and bed being cultivated, came down in an unprecedented flood. With the exception of one railway bridge, which was founded on deep sunk iron caissons, every single bridge on this long watercourse was destroyed; the aqueduct also was washed clean away and disappeared completely from sight, being buried in an immense hole scoured out owing to the obstruction. The loss of this aqueduct was a most serious disaster, as, being situated not far from the head work, the irrigation of the greater part of the canal was completely closed, involving great loss of revenue and interest on the capital charges. A new aqueduct near the old site was immediately put in hand, the design being entirely recast to provide for passing 175,000 cubic feet per second. The new design consists of 15 spans of 60 feet, the piers, abutments and wings being founded on circular wells 20 feet in diameter, sunk 50 feet below the bed of the stream. The structure is not provided with a floor, such indeed being unnecessary owing to the great depth of the pier foundations, which are below the influence of any scour.

**(10)** As already noticed, the flood discharge of the Kali Nadi has been estimated, as far as can be gathered from the meagre information attainable on the subject, to be close upon 175,000 cubic feet per second (*vide* Vol. XC.,



Figs. 3, 3a, 3b, 3c.—Kali Nadi Aqueduct, Lower Ganges Canal. Fifteen Spans of 60 feet.

"Min. Pro. Inst. C.E."). The data on which this estimate is based appear to be very rough. The normal velocity of the stream has been taken as 3 feet a second only. This, if not a mistake, can only be accounted for by the depth of the stream being greatly increased by scour of the sandy bed, and further by the water in motion being very heavily charged with floating particles of silt; and yet in the calculations for the discharge, this increase in the depth due to scour of the normal bed has not apparently been recognised.

The area of clear waterway in each span is 1,140 feet, and there are fifteen spans—total, 17,100 square feet. This area has to pass 175,000 cubic feet, hence the velocity of current passing the bridge will be $\frac{175,000}{17,100} = 10\frac{1}{4}$ feet per second. This immense velocity would speedily scour out the bed till equilibrium was produced. In the original project it was proposed to place a floor 10 feet below the normal bed level of the stream to allow for scour down to this level. This would enlarge the area of the waterway to 24,000 square feet and decrease the velocity to $7\frac{1}{4}$ feet per second. This velocity again is excessive, and if the floor as proposed were actually constructed, which is doubtful, further scour must necessarily occur in the unprotected river bed immediately below. This hypothesis is borne out by the recorded action of subsequent floods, one of which is stated to have scoured out a hole 30 feet deep below the work. From the above it is evident that the depth of pier foundations designed, viz., 50 feet, is in no way in excess of requirements.

**(11)** With regard to the superstructure, as the flood rises as high as the bed level of the canal, the arch and spandrels of the bridge openings cause considerable obstruction to the waterway. This obstruction, exclusive of the piers, is no less than 400 square feet in a water area of 1,600 feet, *i.e.*, one-quarter of the area of the waterway above the normal bed level of the river. The obstruction due to the pier cutwaters is not included in this estimate. This constriction of the passage is unavoidable in the arched type of superstructure in all cases where the flood level is higher than the crown of the arch, but the adoption of a steel girder superstructure for the aqueduct would very largely reduce the obstruction. In that case the only obstruction below the water line would be the joists and longitudinals below the plate floor; these would not take up more than 2 feet in depth, and the proportionate obstruction would be only one-twelfth of the normal waterway. However, under the peculiar circumstances of this particular case the relief thus afforded would be comparatively small. The congestion occurs mainly in the base of the water section, which is relieved by scour of the bed at each flood. Consequently the necessity of deep isolated foundations to the piers would still remain, as also the inadvisability of attempting to curb the bed scour by the imposition of a floor, even if sunk below the normal bed.

**(12)** In rivers, whose bed is composed of micaceous sand of low specific gravity, as the Ganges, Jumna, and the Kali Nadi in question, the bed level is being constantly altered by flood action, the water section automatically adjusting itself to varying discharge and velocity. During the prevalence of high floods the sandy bed is scoured out and carried along by the current, and this process continues until equilibrium is produced by the fall of the flood. Then with the consequent decrease in the velocity, the silt in suspension begins to deposit and fill up the bed, till eventually the normal low water bed level is again reached. This action takes place in all rivers, but is most marked in those which run in soil of a very light character. In bridging a river of this description the only safe plan is the adoption of very deep isolated pier foundations, generally associated, in consequence of the expense of these foundations, with wide spans.

**(13)** In the design under review the deep foundations are rightly provided, but owing to the arched type of superstructure adopted, the spans have to be limited in width; consequently the foundations of the piers are too close together for economy. If laid flat, the well foundations would form a continuous solid floor nearly 20 feet in depth. Consequently in this and in all similar cases wider spans are a distinct economy, to effect which a steel girder superstructure is a necessity. Such an aqueduct would not cost more than a railway bridge of similar span and width, because the load carried, although greater in amount than in the former case, is practically entirely dead load, and the aqueduct would probably be even a lighter structure than a railway bridge subject to a heavy vibrating live load. By adopting moderate spans of, say, 120 feet, the expense of pier foundations could be reduced by one-half, or nearly so; while, on the other hand, the safety of the work would be considerably increased by the lessened obstruction of the waterway and consequently diminished scour, which would be decidedly a desideratum, as a 30 feet deep scour, previously alluded to, can by no means be regarded with equanimity even with the 50 feet deep foundations adopted. The length of the aqueduct piers with a girder superstructure would have to be increased, as the canal water section would be split up into several channels, each separated by the girders, the vertical box plates of the lower flanges of the latter being carried up to above full supply level to form the sides of the channels, the rest being open truss work. With eight spans of 120 feet, the available waterway above normal bed would be $8 \times 120 \times 22 = 21{,}120$ square feet. The velocity of passage before the bed is scoured out would then be $\frac{175{,}000}{21{\cdot}120} = 8\frac{1}{4}$ feet per second, and at 10 feet lower level $\frac{175{,}000}{30{,}720} = 5\frac{1}{2}$ feet per second nearly. This allows of an increase in the width of the waterway as executed, which is evidently desirable.

**(14)** This increase in width must, however, be limited so as not greatly to exceed the normal width of the river channel. Scouring action in the bed of a river cannot be prevented by greatly widening the natural channel and lengthening the bridge. If this is done in the mistaken idea that the natural scour of the bed would cease, owing to the provision of ample waterway laterally, the certain action of the first falling flood would be to deposit silt on both sides, thus reducing the channel to its normal proportions. In future floods this lateral deposit would either remain undisturbed—the channel reverting to its normal regimen—or else the current would bore it out on one side, heaping deposit on the other, thus causing great danger to the work from cross-currents. This property of rivers is well known, and in the Punjab, for this reason, the bridges over several large rivers have had to be curtailed in length and the channel restricted by protective works into a straight, comparatively narrow reach, more in accordance with the normal channel.

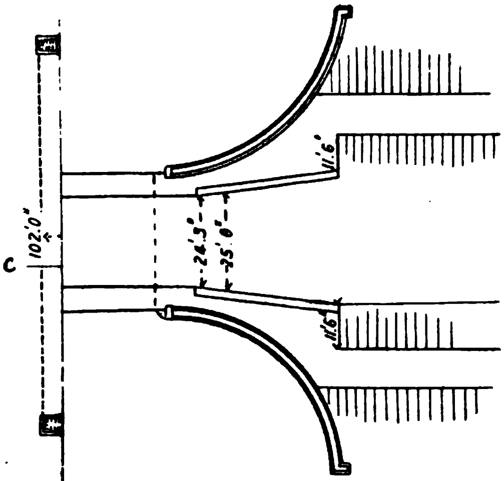
(**15**) We will now proceed to review the details of this work. Excepting the sections of the wings, which, as is invariably the case in Upper India, are designed with vertical faces, the details of this work are excellent. The arch is $3\frac{1}{2}$ feet thick at the crown, or $\cdot 42\sqrt{r}$, increasing to 4 feet at the springing.

The thickness of the piers at springing is 7 feet, or $\cdot 116S$, or $\frac{S}{8\frac{1}{2}}$. This thickness must be considered as below the average. As $\sqrt{S}$ the thickness would be 7·7 feet. The piers widen out in a curve to 9 feet at the base. The spandrels of the arches are lightened by a series of jack arches supported by the longitudinal piers shown in Figs. 3 and 3a. The parapets are corbelled out to form the roadways on either side. One provides a 12 feet cart track, and the other a 6 feet wide footway. The corbelling and arching is ingeniously arranged, but it takes up a great deal of room, necessitating a lengthening of the arch, piers and abutment, which could be reduced, if no regard were paid to architectural features.

The wings consist of a pair of curved landings ending in the canal earthen banks, which are here of semicircular shape to receive them. On the river side are two large water wings, curving a full quadrant and continued to well beyond the termination of the land wings, forming a very efficient protection to the flanks of the work.

The discharge of the canal is 4,100 cubic feet per second, which has a velocity of 4 feet per second in passing the aqueduct; the curves narrowing the canal are not shown on the plan; the ordinary bed width of the canal is 230 feet. The banks on each side of the narrowest part are revetted, and enclose a puddle core of large dimensions.

(**16**) Figs. 4 contain the plans of the Gunneram Aqueduct over a branch of the River Godaveri, taken from the "Madras Irrigation Manual." It has forty-eight spans of 39 feet, and the width between parapets is $23\frac{1}{2}$ feet, with a depth of water of $6\frac{1}{4}$ feet. The length of this structure, which is 2,250 feet between abutments, or nearly half a mile, is such that a slope is necessarily given to the canal bed of 18 inches, the spring line of the arches being built lower in each span. The surface level of canal, however, shows only a slope of 6 inches (*vide* sections on $AA$ and $CC$), which would be ample.



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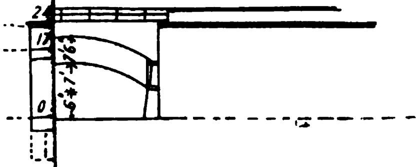

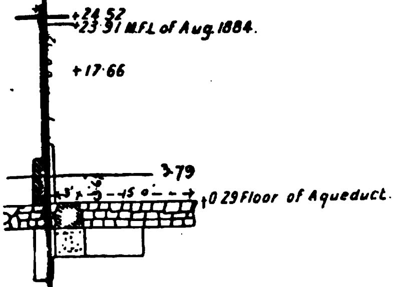

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The river flood has risen as high as R. L. 23·91, *i.e.*, nearly 24 feet above bed, and actually as high as the surface of the canal supply in the aqueduct (*vide* section on *AA*). This must have subjected the work to a very severe test, and clearly proved the sufficiency of the shallow foundations adopted. The piers and abutments are built on circular wells 6 feet in diameter and 7 feet deep, leaving an interval of 3 feet to floor level, which is occupied by the spreading bases of the piers. A continuous floor of pitched rubble 3 feet deep, protected by a series of horizontal curtain walls founded on shallow wells, the whole 6 feet deep, is provided. The existence of this floor is what undoubtedly prevented the pier foundations from being undermined during the exceptional flood alluded to. Deep iron or steel sheet piling fore and aft of the floor would, however, make a better protection with mass concrete filled in between.

Owing to the great difference in the material of which the respective river beds are composed, no useful comparison between a work, which is the exponent of shallow foundations and the Nadrai Aqueduct can well be made. Under the conditions prevailing in the case of the Kali Nadi, shallow foundations, as have proved so successful in the Gunneram Aqueduct, would undoubtedly be a failure, owing to the great velocity engendered and the non-resisting character of the material of the river bed, unless they were continued for a very long distance below the work and reinforced by sheet piling. Such a construction, though it would interfere considerably with the regimen of the river, would doubtless stand, but its cost would probably be equal to that of the deep foundations adopted, in addition to which the point of dangerous scour would only be transferred lower down the river bed.

(**17**) As regards details, the parapets and spandrel walls are remarkably slight, being only 2½ feet thick. The spandrels are, however, filled up solid with concrete to the level of the intrados of the arches. The widening of the parapet to form a roadway is effected by external arches springing at a higher level, which level is horizontal throughout, not on a falling incline, as is the case with the main arches. These arches are carried by stone columns built on the cut-waters. This projection is 3 feet wide, allowing a top width of 5½ feet for the roadway on either side. A better arrangement, and much lighter, would have been to have made the parapet walls thicker, say 4 feet, and used iron brackets for the widening at top. The thickness of the arches at the crown is 2 feet 9 inches, or $\cdot 45 \sqrt{r}$.

The piers are 6 feet wide at the spring line, or $\cdot 15 S$, and widen out to 7 feet at bed level, and to 10 feet at top of the well foundation. These appear to be in good proportion, $\sqrt{S}$ being 6·32 feet. The section of the abutments is very light, due credit being given to the large buttress in the centre.

These abutments, with 40-feet spans, are of four-fifths less sectional area than those of the Kerai Aqueduct, with only 20 feet spans, the absurdity of which has been already noticed when reviewing the details of that work.

The disposition of the wings is peculiar. There are two large concave level crested wings, which start from the outside of the abutments, forming water wings, inside which is the earth of the embankments, while the water is apparently contained by two inner walls, which are not shown to have any foundation, and may be just vertical walls of dry masonry.

This arrangement is probably due to there being no natural banks to the river, the canal embankment itself confining the river within the bridge abutments, and these run at right angles to the axis of the stream. Without a cross-section at this point the exact arrangement of the junction of the earthen bank with the masonry of the bridges cannot be ascertained, the plan over-all being certainly defective. This work is contemporary, or nearly so, with the Solani Aqueduct on the old Ganges Canal, having been finished in 1852.

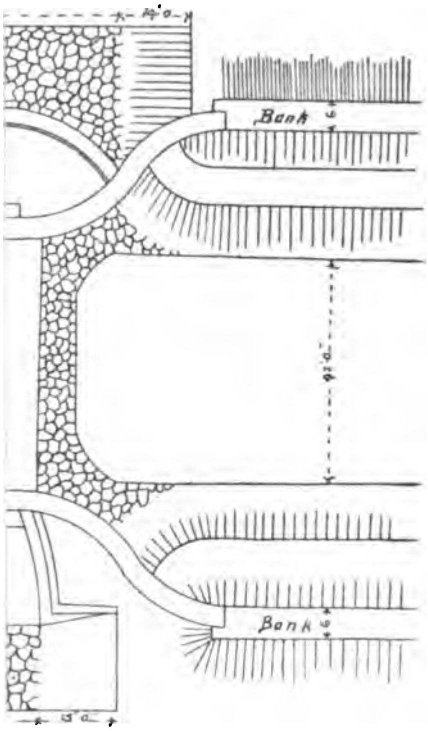
**(18)** Figs. 5 contain the plans of the Kesarapali Aqueduct on the Ellore Canal, Madras. This work is rather remarkable in having a clear overfall of 12 feet in the bed of the drainage line; the design thus resolves itself into a bridged fall, with the bed of the canal below that of the upper reach of the drainage line.

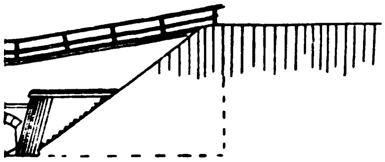
A depressed bed involving a so-called syphon has been avoided, which is always to be effected if practicable, as syphons are very liable to fill up with detritus and cause a breach in the canal. Another marked peculiarity in this design is that the crests of the parapet of the aqueduct are purposely kept only just above F.S.L.; this is to enable surplus canal water to spill over either side of the aqueduct into the drain. The work then fills the *rôle* of an escape as well as an aqueduct.

The openings are 10 feet wide by 5½ feet high, and are provided with inverts which take up a great deal of waterway and could well be dispensed with, the floor being made thicker, if necessary. There is little or no upward pressure on the roof, so that arched vaulting is suitable. The design as a whole and in detail must be pronounced excellent.

**(19)** Figs. 6, 6a and 6b are a representation of the Kao Nadi Syphon Aqueduct carrying drainage underneath the Sôn Canal. Both halves of the work are exactly similar. In order to save headway the tunnels are vaulted with flat ashlar slabs, which are held down by bolts passing through the piers, and are covered with a layer of brickwork and concrete. A 10 feet roadway is provided on each side, consisting of slabs of stone resting on stone corbels.

**(20)** The flood level of the outside water is R. L. 330; that of the underside of the slab roof is 319. There is consequently a head of 11 feet of water acting upwards, tending to lift or break the roof when the canal is empty. When the canal is full and the drain empty, the weight of water pressing on the slabs is that due to 9½ feet depth of water. The former pressure is the greater, but in calculating the thickness required for the slabs the weight of the slabs and the superimposed masonry and concrete is in their favour, whereas in the latter case it is against their resistance to rupture, so that the slabs are subjected to a uniform load of, say,





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10 feet of water plus their own weight. Taking this at a specific gravity of $2\frac{1}{4}$ and the thickness being $2\frac{1}{2}$ feet, the load is equivalent to a weight of $10 + (2\frac{1}{2} \times 2\frac{1}{4}) = 16$ feet of water weighing $\frac{16}{36} \times 20$ cwt. = 9 cwt. per square foot, the span, or $L$, being 6 feet, or 72 inches, the width $B$ being 12 inches. To find the required depth, using a factor of safety of 4, the breaking weight, or $W$, is 36 cwt. per square foot. Then the following formula for strength of rectangular beams uniformly loaded, taken from "Molesworth Pocket Book," p. 136, will state the case, viz., $W = \frac{8KBD^2}{L}$. This involves two unknown quantities, $D$ and $K$, of which $K$, *i.e.*, the coefficient of rupture, must be obtained by experiment. This is easily done by having some beams, say 1 or 2 inches square, cut off the stone to be used, laid out on supports at a convenient distance apart, say 4 feet, and loaded in the centre by a hook passed over carrying a weighing platform on which weights are placed by degrees till the beam breaks. The weight of the platform, etc., should naturally be included in the count. Then $K$, or the modulus of rupture, will equal $\frac{LW}{4BD^2}$ either in inch lbs. or inch cwts. as $W$ is taken in lbs. or cwts. In this case $L$ is the length between supports of the beam operated on, $W$ the average breaking weight of several experiments, and $B$ and $D$ the breadth and depth of the beam placed under loading. Having thus obtained a reliable value for $K$, it can be substituted in the formula $W = \frac{8KBD^2}{L}$, whence $W$ being four times the distributed load on the slab 1 foot wide in cwts. or lbs., $B$ being the breadth 1 foot or 12 inches, and $L$ the length between the piers, likewise in inches. $D$, the required depth, will be $\sqrt{\frac{LW}{8B}}$.

**(21)** In this example the levels of the inlet and outlet of the syphon are the same, hence the water will be higher one side than the other to allow for the increased velocity which is generally allowed for, in designing these works.

In the case in question let us assume, for example, that the flood discharge of the Nala is 2,400 cubic feet, the waterway having an average width of 100 feet and depth of 8 feet, as shown in Fig. 2, and a mean velocity of 3 feet per second.

Let the limiting velocity in the syphon be 5 feet per second then each opening is capable of discharging $36 \times 5 = 180$ cubic feet per second, and thirteen openings will be required. To calculate the heading up to produce this velocity it is well to leave the velocity of approach out of consideration, as it will be neutralised by the heading up. Now the head to produce this velocity will be $\cdot 0155\ V^2 = \cdot 0155 \times 25 = \cdot 388$ or 5 inches nearly.

**(22)** This, however, does not represent the actual state of affairs, unless the outlet channel below the work is improved so as to be able to carry the increased discharge. For the discharge will be increased above the maximum for the following reason, viz., that the heading up of the stream caused by the obstruction will increase the sectional area, and although the supply remains the same yet the accumulation of water and consequent raising of the head will temporarily cause an increase of the discharge until the normal level is reached. If the channel on the far side of the work is not capable of carrying off this excess discharge without an appreciable rise in the water level, there will be a certain heading up on the down stream side as well, which will reduce the head of water correspondingly and consequently the discharge capacity of the syphon, thus necessarily further increasing the head upstream. Thus we see that unless the lower channel can be improved in carrying capacity it is not well to reduce the sectional area of the syphon below what is required to carry the flood discharge through at the normal velocity of the stream. In the example we have given above, the discharging capacity of each vent should be $36 \times 3 = 108$ cubic feet per second, and the number of vents $\frac{2,400}{108} = 22$.

**(23)** We have seen that the upward pressure below the slabs is that due to a head of 11 feet of water; from this should be deducted the weight of the slabs and superimposed material equal to a corresponding weight of water per square foot of $2\frac{1}{2} + 2\frac{1}{4} = 6$ feet. The balance of head acting will then be $11 - 6 = 5$ feet of water or $\frac{5}{36}$ tons per square foot. This will have to be taken by the tie rods. Supposing the ties to be 5 feet apart, the tension on each will be $5 \times 6 + \frac{5}{36} = 4$ tons nearly (the length of the slabs being 6 feet), requiring a sectional area of $\frac{4}{5}$ square inches, or $1\frac{1}{8}$ inches diameter.

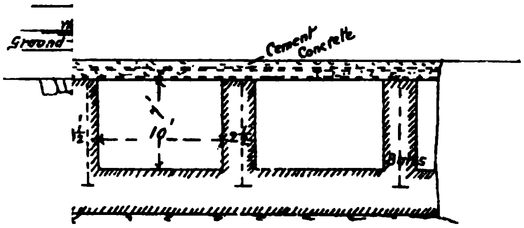
**(24)** The disposition of the wings and the narrowing of the waterway of the canal is shown in the general half plan (Fig. 6b). The land wings, as usual, are curved on plan, which in the case of aqueducts is undoubtedly the only suitable form, while two straight flank walls with sloping crests enclose the cistern at the head and tail of the syphon. The vertical drop at the exit end is a bad form, as it prevents detritus being washed through the syphon, necessitating its removal by excavation. The exit end should not be a dead wall but an easy incline, as shown by the dotted lines in Fig. 6. In most modern designs this sloping approach is provided at the inlet end as well as the outlet of the syphon, as exemplified in Fig. 9, though it is not absolutely necessary at the upper end. Fig. 7 is an example of a vertical drop on the up-stream side, and an incline on the down-stream side of a canal syphon.

**(25)** The drawback to a slabbed culvert of this description is the limit it imposes on the span of the openings and the great expense of the ashlar slabs.

A far better construction would be to form a flat concrete roof supported by rolled iron beams, or of reinforced concrete; the span could then easily be increased to 10 feet, and the cost would be very much less.

Fig. 6c shows the construction.

The beams are 10 feet span and 5 feet apart. The weight on them will



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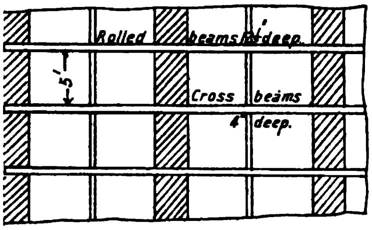

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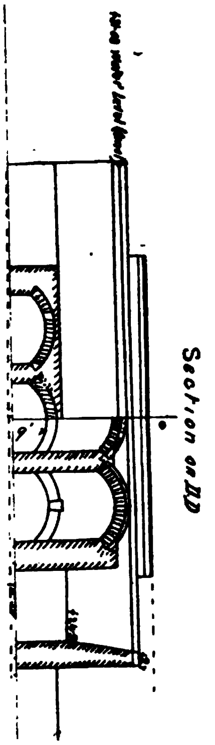
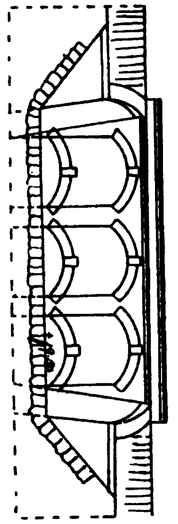


Section on BD



Sec

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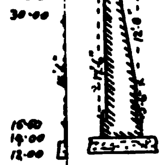
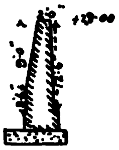
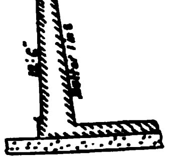


Section on A-A.



Section of Rear Wing at
Junction with Face



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be equivalent to 15 feet of water or the total distributed load = $10 \times 5 \times 15 \times \frac{20}{36}$ = 417 cwt., the weight of a cubic foot of water being $\frac{1}{36}$ of a ton. From the Table given in "Molesworth's Pocket Book," p. 146, a beam 12½ inches deep with 6 inch flanges and $\frac{11}{16}$ inch web will safely carry this load.

Cross-beams of small section will be laid along the centre of the span between each main girder, thus dividing the platform into five feet squares. The total depth of the roof can be reduced from 2½ feet to 1½ feet, giving an additional 1 foot in the headway of the sluices.

**(26)** Another instructive example of a syphon passing drainage underneath a canal is given in Fig. 7. In the transverse section it will be seen that there is a fall in the syphon of 3½ feet. This will enable a large discharge to be passed at increased velocity through the work, the waterway being considerably curtailed to what would otherwise be required. The spans are 9 feet, the piers 2½ feet thick, the headway 6 feet, and the floor 3½ feet thick. The abutment is of reasonable dimensions. The disposition of the wings is apparently good. The earth lines, so necessary to form a proper idea of the suitability of the wings, are not given.

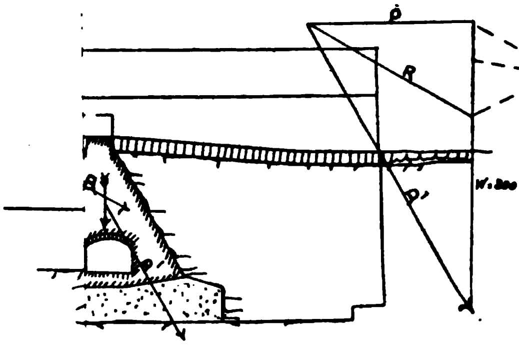
The canal is 7 feet deep and the parapets are 3 feet 6 inches wide at base and 2 feet 6 inches at top. The roadway is formed by an outer archway of 6 feet in width, the piers being lengthened to carry it. This provides a roadway of 7 feet. The exit side of the work has a sloping rise to the bed level of the stream, which is as it should be. The head producing upward pressure is 4 feet. The arch, with concrete covering at the crown, is 2 feet thick, and with a specific gravity of two will just balance this pressure. No exception can be taken to any of the details, except possibly that a thicker parapet corbelled out on both sides would provide an equally wide roadway, and thus save the lengthening of the piers and the external high level arch.

**(27)** Super-passages, where the levels are such that rivers have to be taken over a canal, are comparatively rare, as they involve a very heavy work if the stream or torrent is large, and so are avoided if possible. They generally occur in the upper section of a canal, the head work of which is situated among the lower hills high up in the course of a river. There are some very large works of this description in the old Ganges Canal. An excellent modern example is given in Fig. 8 of the Budki super-passage on the Sirhind Canal in the Punjab.

The length of the work is seven spans of 30 feet, the width between parapets 400 feet. The arches have to be raised well above the canal bed on account of the exigencies of navigation, and also of the level of the bed of the torrent. The 30 feet wide spans are therefore suitable. The thickness of the arches at crown is $\cdot 45 \sqrt{r}$. The thickness of the piers at springing is 6 feet or $\cdot 2S$. We have already seen that a proportion of $\sqrt{S}$, giving a thickness of 5½ feet, is sufficient. The piers are therefore too thick. The bottom width could be retained at 8 feet, the piers having either a straight or a curved batter as has been designed. The foundations, which are probably on good soil, are admirable (*vide* par. 11, Chap. IV.).

(**28**) The abutment, by the look of the section, is clearly much too heavy, a very common fault among designers. To prove this the actual incidence of the resultant line of pressures on the base has been graphically found, the method of working, which has already been fully explained in Chap. IV., consists in first finding the centre of gravity of the half arch and its load of water, the latter reduced in depth, as shown by the horizontal dotted line, to an equivalent mass of masonry. This process is shown in Fig. 8b and the reciprocal funicular polygon above Fig. 8a. The forces 1 and 2 are the areas of the two halves into which the half arch has been divided. Having found the centre of gravity of the half arch, a horizontal line is drawn through the centre of the arch crown to intersect the vertical through this centre of gravity, and from the point thus found the line $R$ is drawn in Fig. 8a through the centre of the arch at its springing, till it intersects a vertical line through the centre of gravity of the abutment and its water load. In the force polygon (Fig. 8b), the load line, composed of the areas 1 and 2, is continued down to measure 300 square feet, the area of the abutment with its load of water, and the line $R$ is then drawn from the termination of 1 and 2, parallel to its reciprocal in Fig. 8a, cutting the horizontal $P$ at a point. From this point another line, $R_1$, joining the termination of the vertical load line 1, 2 and 3 just obtained, gives the final resultant $R_1$. This projected on the profile of the abutment in Fig. 8a from its proper starting point, viz., the last intersection found, cuts the base of the abutment at a point some 5 feet within its heel. As no credit has been given to the weight of the earth backing with water above it the resultant line $R_1$ need only just fall within the base. This proves that the abutment is unnecessarily thick. The line $P$ represents the horizontal thrust of the arch, and if measured, will be found to closely correspond with $rt$ or $8 \times 30 = 240$, *i.e.*, the calculated horizontal thrust of the arch (*vide* par. 23, Chap. IV.). If the abutment were made 8 feet wide at the springing and 13 feet at the base it would probably be of sufficient section. As built, it is half as wide again as these dimensions, and besides is provided with a large buttress of which no account has been taken.

(**29**) It might here be mentioned that the calculation of the effect of buttresses in an abutment or retaining wall is effected as follows:—The wall should be considered as having a base equal to its normal thickness plus the length of the buttresses, but formed of two materials of different specific gravities, the solid portion being of the proper specific gravity of the material and the part behind of a lighter specific gravity, equivalent to that of a material spread over the space of the same weight as the solid buttress only. Thus, supposing a wall is 6 feet thick and is provided with buttresses projecting 4 feet and 4 feet thick and 6 feet apart, *i.e.*, at 10 feet intervals, and let the specific gravity of the wall be 2, then the specific gravity of the 4 feet wide space behind will be $2 \times \frac{4}{10} = \frac{4}{5} = \cdot 8$, and the effective base width of the wall be 10 feet, not 6 feet.



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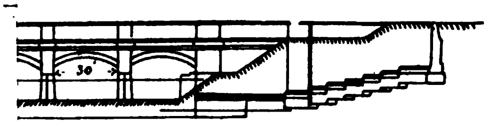

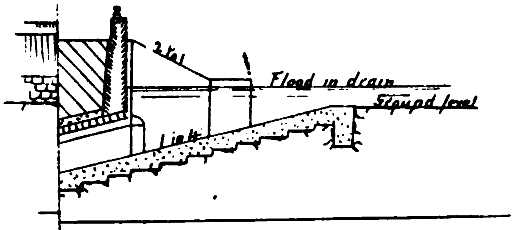

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(**30**) The disposition of the wings is generally similar to that usually adopted in aqueducts, consisting, viz., of water wings as curved continuations of the faces of the abutments, and splayed land wings which carry parapets in continuation of those in the aqueduct proper. These wings are shown in the plan over all (Fig. 8c). As a further precaution, the ends of the land wings are connected by a cross wall at bed level, which apparently goes down to the full depth of the foundations. The land wings being in solid ground are stepped up in foundation, which is shown in the elevation Fig. 8d.

(**31**) Fig. 9 shows longitudinal and cross section of the type design for syphons crossing drainage, adopted in the Chenab Canal, which, being the most recent large irrigation work in India, will probably contain better designs than those on the older canals. It will be noticed that the canal banks are carried right across the syphon with a pitched slope on the water side and a revetment wall outside. This is in order to admit cart communication across the aqueduct, which is always a necessity. This arrangement for a low culvert, two or three spans only in width, is probably the most economical, as it does away with the necessity of any land wings and of additional arches to carry the roadway, as adopted in Fig. 7. The upward head of water below the crown is 9 feet. To balance this a thickness of roof of $\frac{9}{\rho}$ is required, taking the specific gravity or $\rho$ as 1·8, the thickness should be $\frac{9}{1 \cdot 8} = 5$ feet. The actual average thickness of the arch and concrete above is about 4½ feet. There is therefore tension in the masonry equivalent to the weight of a layer of brickwork, etc., of ½ foot in thickness, which in lbs. is $\frac{1}{2} \times 1 \cdot 8 \times 62 \cdot 4 = 56$ lbs. per square foot. A few tie rods are thus required to carry this unbalanced stress, which otherwise would exert a transverse upward stress on the arch, which, from its construction, it is least adapted to bear. No doubt the cohesion of the mortar in the concrete above is more than sufficient to withstand this pressure, but in all masonry works, particularly those in connection with water, any tension should be avoided.

(**32**) The general arrangements in this design are simple and excellent. The floor is only 2 feet thick, and the whole is made entirely of concrete. The proportionally wide spans are also a good point, as the wider they are the higher the coefficient of discharge will become. The double slope is a good arrangement for drainage lines, as it gives an impetus to the water and tends to carry detritus through the culvert. A direct overfall on the upstream side, on the other hand, absorbs a great deal of the velocity of approach. The form of vertical drop above and slope below is suitable where a canal is taken under a drainage line, as being more economical.

(**33**) Distributaries and branch canals are often taken by a syphon underneath a low roadway or drainage line. In the former case an overbridge is avoided as also the interception of the drainage that generally passes down a road. An example of such a work is given in Fig. 10, which represents a notch overfall weir with a low level bridge or culvert beyond, through which the water passes. Fig. 11 is a design for a similar work, but without any drop in the bed; the culverts in this case act as a syphon.

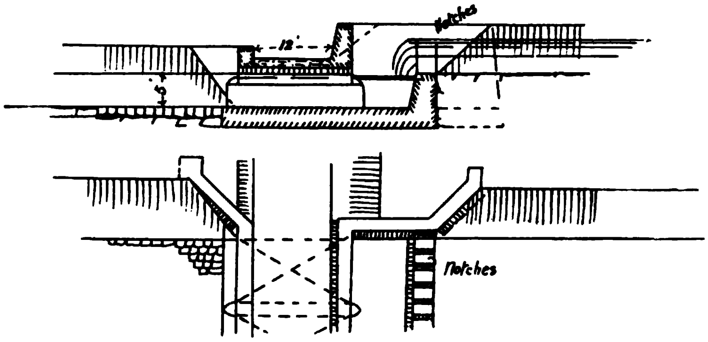


Fig. 10.

In such cases it is usual to give a small drop to prevent any heading up. In this instance the notches are not carried up the full depth of the canal water, but only half-way, to allow a free passage when the syphon is full.

(34) Two Egyptian works are illustrated in Figs. 12 and 13. The first is a plan of a syphon under the Sohagiah Canal, taken from "Egyptian Irrigation." It has a vertical fall up stream and a slope down stream.

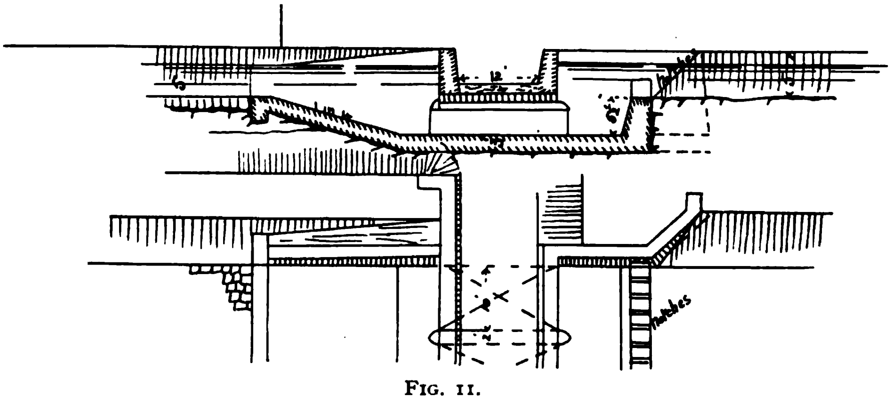


Fig. 11.

The fall is provided with grooved piers and a plank platform, possibly with a view of holding up the water to cause a scour when the stream is in low discharge. The well between the weir wall and the face of the syphon is of very great width, viz., 14 metres, or nearly 50 feet. This appears quite unnecessary. A width equal to the depth of water, *i.e.*, of about 10 metres, or 30 feet, would be ample. There does not appear to be any fall in the bed of the drain. The openings are about 10 feet wide and 7½ feet high, the piers 3¼ feet thick, which is somewhat excessive. The thickness of the roof is 2 metres, or 6½ feet, which is sufficient for a head of 13 feet. The flood level of the drain is not shown, but from the height of the banks it is probably about R.L. 61 at least. That of the intrados of the culvert arch is 54·50; the head then will be 6½ metres, or 21½ feet. The syphon is evidently not designed to run full with the canal empty, as it undoubtedly should be. This state of things is mentioned by Sir Wm. Willcocks,

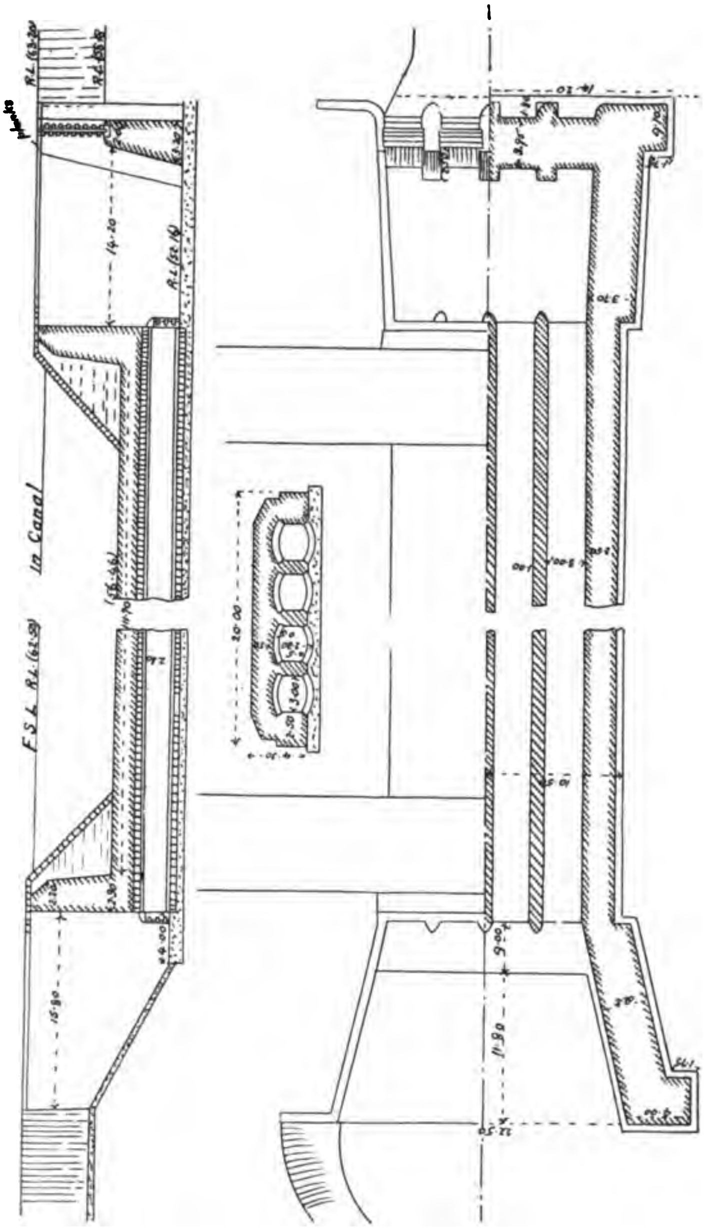


FIG. 12.—Sohagiah Canal Syphon (Egypt).

in "Egyptian Irrigation," many of the older works in Lower Egypt having been designed on the hypothesis that the canals would never run dry, *i.e.*, the supply would never be cut off from the head. Now, however, that head regulation is possible, these works are not equal to the pressure brought upon them, and this doubtless is one of them.

(**35**) The section of the vents could easily be enlarged to great advantage, by adopting the system of a thin concrete roof, reinforced by iron beams or rods and bolted down as shown in Fig. 6c.

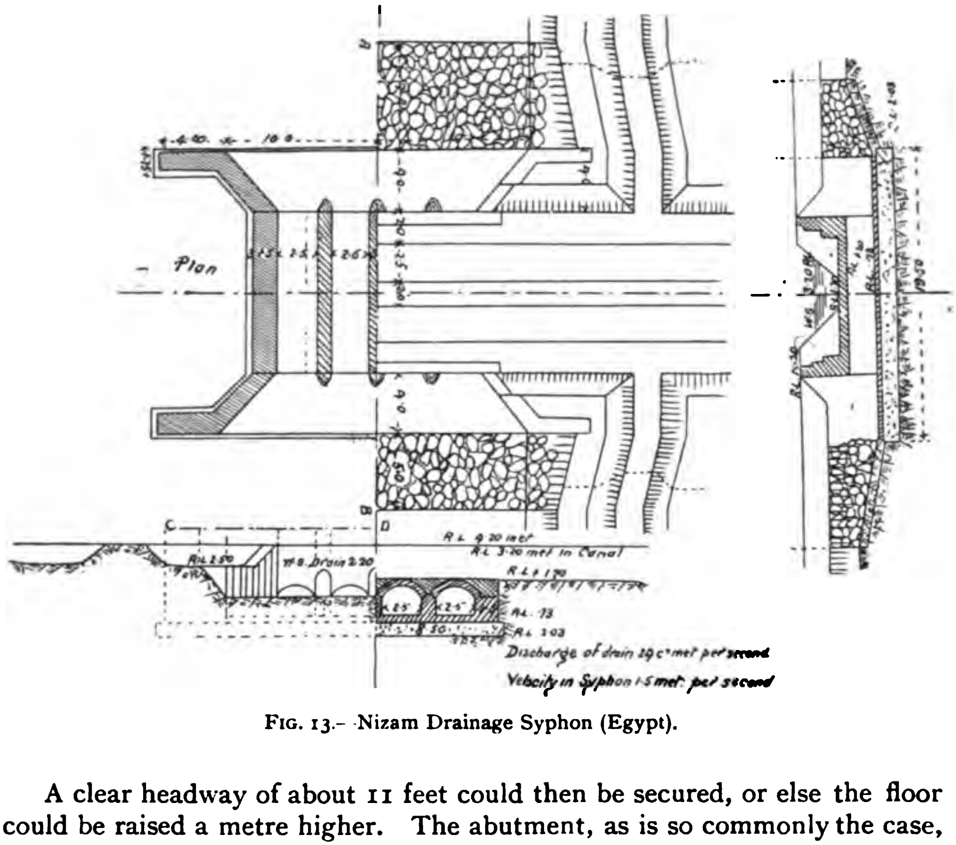


FIG. 13.– Nizam Drainage Syphon (Egypt).

A clear headway of about 11 feet could then be secured, or else the floor could be raised a metre higher. The abutment, as is so commonly the case, is unnecessarily thick. The slope of the earthen banks is carried through the work. This saves land wings, but it is doubtful if any economy accrues considering the lengthening of the heavy syphon barrel which this system involves. On a small work it is doubtless suitable, but this is a very large work, though it does not appear so from the drawing, the scale being small. The distance apart of the parapets of the aqueduct is no less than 350 feet, and it carries 20 feet of water.

(**36**) Fig. 13 is another Egyptian syphon, but of quite small dimensions. This is provided with a double slope, which is of pitching. The disposition of the wings and the banks are given in the plan over-all, which is most useful as a guide in similar cases. In this design the banks of the canal are carried right across. The outer parapet walls are of too heavy a section, being nearly as wide at the base as the depth of water in the canal. The arches of the barrels might well have been made flatter.

(**37**) Fig. 14 is a type section of a syphon under a distributary from the "Madras Irrigation Manual." As will be seen, both earthen banks are carried entire across the syphon ; the general arrangement is simple and effective.

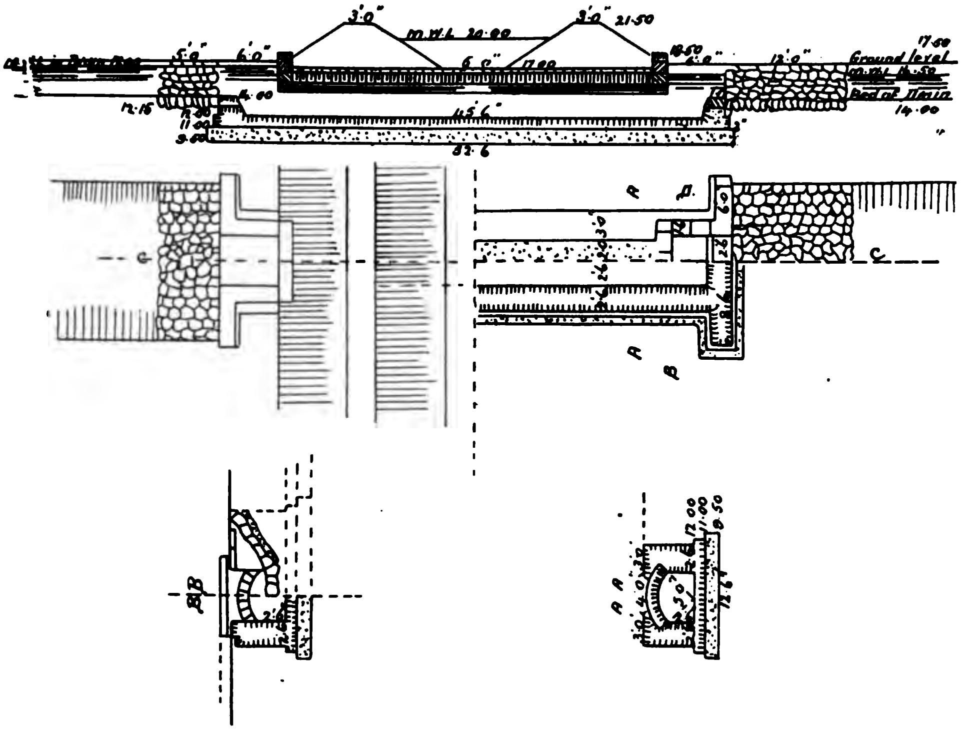


Fig. 14.

(**38**) Fig. 15 is a type diagram of a super-passage on a small scale taken from the same source as Fig. 14. This is provided with land and water wings of the usual Madras pattern, the battered section of which is a vast improvement on the unscientific vertical faced revetment walls, so absolutely universal in Upper India and Egypt. The design appears a good and economical one.

(**39**) In Egypt of late years wrought iron pipes have been used for both aqueducts and syphons. The following is a description of them from "Egyptian Irrigation":—

"They are generally constructed of ¼ inch sheet iron, butt jointed, stiffened with angle irons at every alternate joint if over 12 feet in circumference, and lap jointed if under 12 feet. Since the sheets in the market are 8 feet by 4 feet, or 6 feet by 3 feet, the pipes are always constructed with their circumference some multiple of the length or width, so that there may be no cutting of plates. The pipes are sometimes laid on a bed of concrete, varying from 1 metre to 25 centimetres in thickness, according to the quality of the soil, or they are laid on the hard clay soil and well pitched

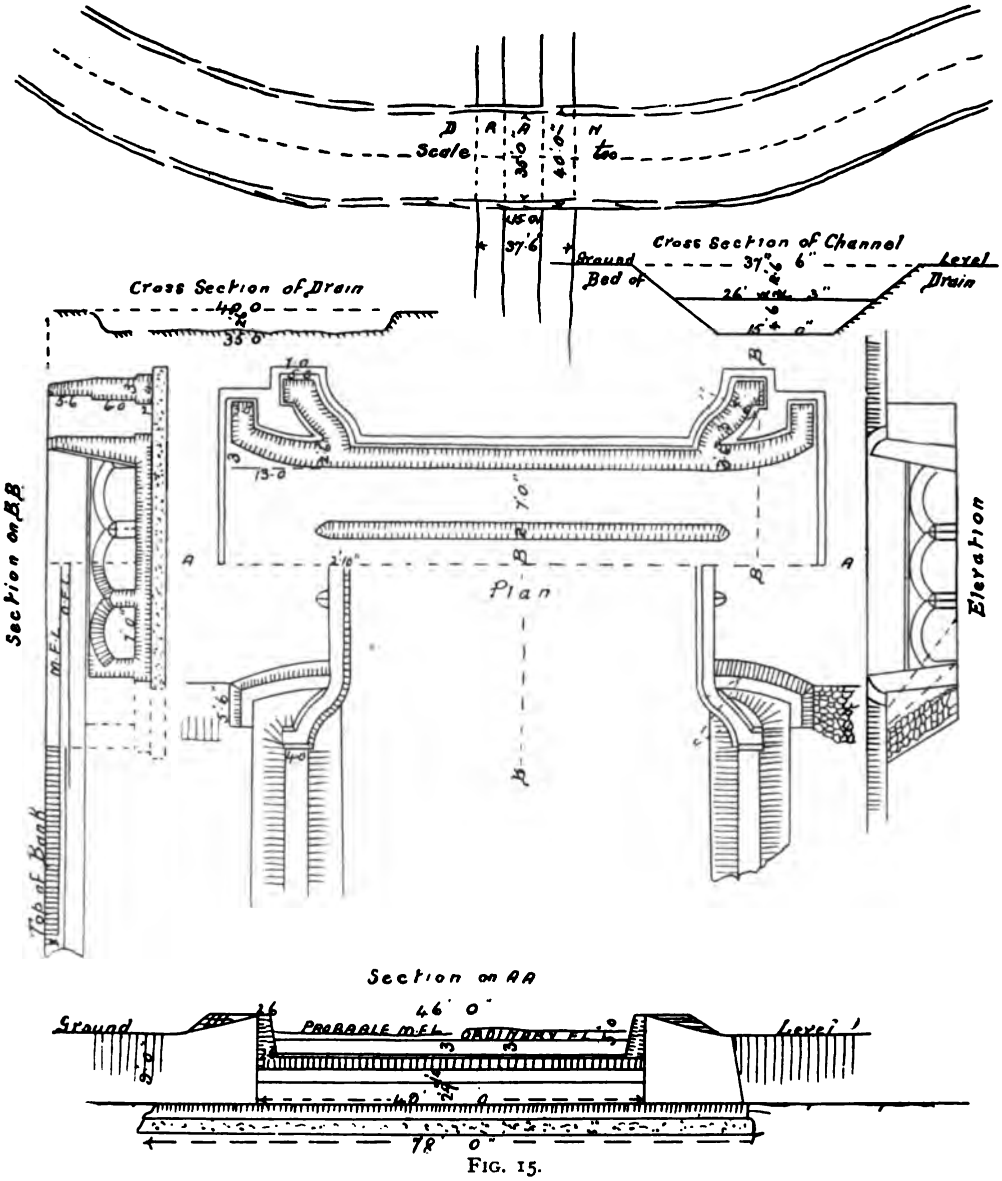


FIG. 15.

round with clay balls. Where the pipes are used as aqueducts they are generally supported on wooden trestles. The great advantage of using wrought iron pipes is that they can easily be transported; they do not need expensive supervision during construction, and can be put together so rapidly that the cost and trouble of a diversion for the canal during the time of construction is avoided. These pipes can be closed at the ends and floated to their destination. By dredging the foundations where they have to be laid they can be floated over the site and then sunk without shutting the canal head."

A pipe of 16 feet circumference will discharge 300 cubic feet per second, with a head of 2 metres or 6½ feet, and 156 cubic feet with a head of half a metre or 1·6 feet. One of 12 feet circumference, 180 cubic feet per second, with a head of 2 metres, and 87 cubic feet, with a head of half a metre.

**(40)** Where the drainage is slight, inlets can be provided; they simply consist of a small fall or pitched rapid, protected by flank walls, which conducts the drainage water into the canal, the floor and banks of the latter being pitched all round to prevent damage. If the inlet occurs at the roadway bank, it has to be bridged.

**(41)** There are no plans of any level crossings available, and it is believed that there is only one example extant, that on the old Ganges Canal, which design is too antiquated to be of any use as a guide. A level crossing consists of an open inlet on the side of the torrent, a regulating bridge across the canal, and another at right angles across the exit of the torrent, which is generally provided with a fall to expedite the speedy removal of the unwelcome guest. It is very essential that quick-acting falling gates be supplied to the outlet bridge; draw gates would be much too slow in manipulation. The balanced pivoted gates recommended for the Koshesha Escape would, it is believed, answer admirably for such a purpose.

**(42)** Before concluding this chapter, mention should be made of cases where the passage of drainage across a canal is beset with the following difficulties:—

(1) The drainage is too extensive to be wholly admitted into the canal.

(2) It cannot be taken underneath in a syphon drain, owing to the presence of the backwater of the parent river when in flood, which comes up to the very canal bank, hindering the free discharge.

(3) It thus must be disposed of by a superpassage; but this involves the formation of a large deep reservoir on one side of the canal, which has to fill up before sufficient elevation is attained to admit of surplus passing at a high level over the canal.

**(43)** This problem was satisfactorily solved in the case of the Ali Super-passage and concomitant works in the third or fourth mile of the

Agra Canal. The works constructed consisted of, firstly, a solid embankment on the upper side of the canal which crossed a large depression; secondly, a water tower built some way off the canal in the depression, provided with external sluice openings at different levels; thirdly, a culvert which connected the base of the tower with an inlet into the canal itself; lastly, an iron girder superpassage with a masonry fall in the further (river side) bank. This arrangement has answered well. When the depression fills up to a moderate extent all its water can be gradually drawn off through the tower into the canal. When, however, as happens not annually but occasionally, the tributary streams bring down so much water as to quite fill the reservoir, the surplus is then disposed of automatically by passing over the super-passage into the river. When the crisis of the flood is over the deep reservoir can be gradually tapped into the canal by means of the water tower and inlet culvert.

# CHAPTER XII.

## DESIGN OF CHANNELS.

(1) THE method pursued in designing a canal irrespective of its masonry works, can only be treated in a general manner. In order to give actual examples of any value for instructive purposes, maps of an irrigation project would be required, with a host of statistics, such as the levels of the country affected, the amount of water available at low and high supply in the river, the irrigable area, besides the cost of the headwork necessary to give the required depth of water in the canal, and many other matters. Such information belonging to an actual project not being available, it would be very difficult to produce imaginary conditions whereby an example could be worked out in detail.

(2) The first point to be decided on is the water available in the river of supply; with this information the designer will be able to form an opinion of the maximum and minimum discharge of the canal—regard being also had to the irrigable area commanded, and to financial considerations.

These being settled satisfactorily, the depth of water in the canal is one of the first points to be decided on, as also the sill level of the headwork with regard to the weir crest and Undersluice floor.

The limiting velocity with full supply will then determine the bed slope of the canal to carry its discharge with the fixed depth, and this slope can be obtained by use of the Tables in Jackson's "Hydraulic Manual."

The bed width of the canal and its side slopes, *i.e.*, the sectional area of the waterway, can be simultaneously worked out.

(3) Main canals have to run for many miles from their source of supply without any irrigation branch taking off: how far depends on the slope of the country. As soon as the bed slope of the canal begins sensibly to gain on that of the country, a fall will be required to keep the canal in cutting, and at such sites branches generally take off. As the canal proceeds, allowance must be made by reduction of the base width in each mile for absorption and evaporation, and the same applies to branches and distributaries.

(4) We have seen in Chap. VI. that the question of the quantity and quality of the silt and sand carried in suspension by the water entering the canal forms one of the principal points to be considered in designing an irrigation channel. In most rivers the silt is of a fertilising character, and consequently it is advantageous to adopt means whereby it can be conveyed

right on to the fields irrigated, instead of being deposited in the channels near the head.

The inclination given to the bed should be such that a uniform velocity is induced right through the system, as any sudden changes or checks in the velocity of the current will immediately cause deposit. The matters in suspension are often composed in part of heavy sand and in part of light fertilising silt. The heavy sand should, if possible, be kept out of the canal. The only partially effective method to bring this about is to keep the lower gates of a head regulator closed during floods, and to admit water only at the top. This will exclude the heavy sand which is pushed along the bed of a river at every freshet. This precaution will, however, but mitigate the evil, which can only be satisfactorily dealt with by constructing a scouring Escape Head a short distance below the canal Head. When more water is available than is required in the canal, the Escape can be partially opened, or when the canal is closed during heavy rainfall, the cross regulator below the escape can be closed and water admitted at the head when it is fairly clear, which will pick up and scour out the deposit on the bed.

(**5**) In cases of low weirs up to 6 feet in height across small sandy streams which are not usually provided with Undersluices, an escape back into the river some short distance below the head is absolutely essential. The channel will otherwise require periodical sand clearance, which is very expensive, and sometimes necessitates the closing of the head when water is perhaps urgently required. When the supply in rivers is intermittent, the stream coming down in occasional freshets and running dry or nearly so between whiles, the Escape can only be opened occasionally, as all the water available is generally required for irrigation, so that in these cases the scour produced should be powerful so as to effect the clearance in as short a time as possible. The banks and bed of a canal thus exposed to scour should be pitched to protect them against erosion.

(**6**) The velocity in a main canal should be kept below that which would cause erosion in the bed and banks; the neglect of this important point has led to immense damage and loss, necessitating the eventual regrading of a canal in California, where the first canals were not designed on scientific principles. According to "Irrigation in India," the following are the maximum velocities allowable:—

| | |
|---|---|
| In light sandy soil . . . | 1½ to 2 feet per second. |
| ,, sandy loam . . . | 2½ ,, ,, ,, |
| ,, ordinary firm loam . . | 3 ,, ,, ,, |
| ,, stiff clay or gravelly soil . | 4 ,, ,, ,, |
| ,, pitched bed and banks, shingle and boulders . | 5 to 6 ,, ,, ,, |

In most Indian canals the maximum velocity does not exceed 2½ or 2¾ feet per second.

The erosive power of water varies with the volume carried and with the amount of silt in suspension. For small volumes, as in distributaries, particularly if silt is carried, the safe velocity can be placed at a higher figure than in main channels of large capacity; thus, if 2½ feet is the maximum allowable in the latter case, an increased velocity of 3 cubic feet per second would probably not be excessive in a narrow distributary, as the scouring capacity of a water section increases with its volume. The maximum velocity adopted should be as high as possible, in order to allow a fair velocity when a lower supply is running, otherwise the channels will become choked with water weeds.

(7) The discharge of a canal or distributary is based on the area irrigable and on the *duty of water*, *i.e.*, the acreage matured by 1 cubic foot per second flowing continuously for a defined time. The duty varies greatly, owing to the losses due to percolation and absorption and to variations in the rainfall in different years on the same canal.

In Upper India there are two distinct seasons for irrigation when different crops are raised. Firstly, the Kharif, that is, the summer season, when tropical crops, such as rice, indigo, cane, cotton, etc., and secondly, the Rabi, or winter season, when crops common to Europe, such as wheat, barley, peas, etc., are raised.

The duty of the Kharif irrigation varies roughly from 50 to 120 acres, and of the Rabi season from 50 to 150 or even 250 acres. The loss of water between the head of a canal and the fields varies from 10 to 20 per cent. under favourable circumstances.

(8) To find the volume of water utilised, the number of days during which the crop is matured is required to be known. This is termed the base of the duty. The volume utilised will then be found by the following formula:—

$V = \frac{B}{D} \times 86{,}400''$, $V$ being in cubic feet (86,400 being the number of seconds in a day of twenty-four hours). Thus, supposing the duty $D$ to be 60 acres, the base $B$ 120 days, then $V = \frac{120}{60} \times 86{,}400 = 172{,}800$ cubic feet of water per acre. The depth in feet will be $\frac{V}{43{,}560}$, or $\frac{B}{D} \times 1{\cdot}51$, or in this case $\frac{120}{60} \times 1{\cdot}51 = 3{\cdot}2$ feet.

This depth can be conveniently termed acre feet, or symbolically $AF$, and, being in smaller numbers, appeals to the eye better than the volume given in cubic feet per acre.

(9) The use of acres 1 foot deep to represent the volume of water used or stored is most convenient in the case of reservoir or tank storage, the cubic contents of which should be expressed in acre feet instead of millions of cubic feet. The capacity will then bear a definite ratio to the area of irrigation to be effected. Thus if rice is the crop to be irrigated from a tank which requires 60 inches or 5 feet depth of water to bring to maturity, and

the effective contents of the reservoir is 10,000 acre feet, then the irrigable area will be $\frac{10,000}{5} = 2,000$ acres.

**(10)** As already noted, canals and distributaries should diminish gradually in bed width to allow for the loss due to percolation and absorption, in addition to the reduction in the required water way due to the taking off of branches or distributaries in the first case, or to that of irrigation channels in the latter case. Thus, supposing a canal is estimated to lose 20 per cent. of its volume in a run of 100 miles; then, neglecting any loss from water drawn off into branches or distributaries, and assuming the maximum discharge at 3,000 cubic feet per second at the head, the water way at 100 miles distant should be calculated for a discharge of $3,000\ (1 - \frac{1}{5}) = 2,400$ cubic feet; at 50 miles, $3,000\ (1 - \frac{1}{10}) = 2,700$ cubic feet; and at 25 miles, $3,000\ (1 - \frac{1}{20}) = 2,850$ cubic feet. With the same bed slope the reduction in water way can be best effected by a corresponding gradual reduction of the bed width.

**(11)** There is no general rule applicable to every case for reduction in discharge due to absorption; each special case must be considered separately.

The author consulted a late Inspector-General of Irrigation (India) on the procedure adopted by him in designing the distributaries of the Punjab canals, and the following are his remarks:—

"There is, as far as I know, no general rule for the deduction of discharge in a channel due to absorption and evaporation, nor, indeed, can there be one. Quite apart from the wetted perimeter come the questions of slope of bed and, above all, the nature of the bed. There are lengths on the Bari Doab Canal, for instance, where little water is lost; again, there are lengths which are like a sieve. The designer must know his ground and apply such lessons of experience as he may have learnt. In any case, I never reduced my sections from above downwards, but always calculated them from the tail end, working upwards, *i.e.*, I *added* from the tail instead of subtracting from the head discharge. My method was as follows:—The area commanded by each distributary was first worked out, and an allowance of 2 cusecs at the distributary head to cover everything was then made for each square mile of commanded area. This gave the distributary discharge, and gives satisfaction in practice. On the branches of the Jhelum Canal I allowed ½ cusec per lineal mile for evaporation and absorption, and 2 cusecs per mile of the main canal. On the Jhelum this latter allowance meant about 1 cusec per 100 feet of wetted perimeter per mile of length, and the allowance has sufficed. In a channel, though, that carries 3,000 cusecs, as in the Jhelum, or say 10,000, as on the Chenab, these allowances are hardly worth consideration, and I do not suppose it would matter much whether they were given or not. On distributaries the allowance of 2 cusecs per square mile amply covers everything in the Punjab. It would, of course, be different in a rice-growing country."

**(12)** To ensure the economical distribution of water, the system of rotation, or, as termed in India, "tatils," has to be enforced. That is, that either the whole of a distributary is closed for a certain fixed number of days in each week, running full supply the rest of the time, or if the distributary is kept continuously running at the head certain lengths of it are closed in rotation. This system enables water to be carried down to the end of a distributary and gives to all cultivators a fair share of the water.

In designing new canals this system of rotation should first be roughly worked out, as the design of the water way is considerably affected thereby. Thus, if the full head supply has to be passed down a closured length, it is evident that the sole diminution in water way will be that due to absorption, and the full or nearly full water way will have to be maintained right through this length. The same will apply to the main canal, if distributary heads are intended to be periodically closed. The length of closure usually adopted in Upper India is five days in fifteen. This subject is well treated in "Irrigation in India," a book which every irrigation engineer should possess, so that it will be sufficient in this work, in order not to "overlap," to treat it with less detail than has hitherto been done.

**(13)** In designing canals from intermittent streams the watering will have to be considerably greater than for a continuous supply.

The following example will explain the method of calculating the discharging capacity of a channel under assumed conditions: Area to be supplied, 2,000 acres; duty of water, 60 acres; then the normal discharge of the channel should be $\frac{A}{D} = \frac{2,000}{60} = 33$ cubic feet per second.

But, assuming that the supply falls short in one month, the river only supplying 33 cubic feet per second during eighteen days, and during the remaining twelve days it can supply an average of 6 cubic feet per second, and let $x$ be the required discharge, then $18x = (30 \times 33) - (12 \times 6) \therefore x = 51$ cubic feet per second.

Again, supposing the river can supply the full discharge for 10 days only in the 30, the average supply of the remaining 20 days being $\frac{1}{4}$ or 8 cubic feet per second, then $10x = (30 \times 33) - (20 \times 8) \therefore x = \frac{750}{10} = 75$ cubic feet.

**(14)** With regard to storage in fields, this must be limited to cases where the freshets occur at intervals of not more than a week, and in which the quantity equivalent to a ten days' supply can be distributed over between six and seven days, while a discharge of $1\frac{1}{2}$ times the normal will suffice for the channel ("Madras Irrigation Manual"). For any greater interval between freshets tank storage will be needed.

# CHAPTER XIII.

## RESERVOIRS AND TANKS.

(1) WHEN water is impounded for purposes of irrigation in a reservoir, it can either be effected by embanking a depression which receives the run off of the rainfall from a catchment area, or else a defined river or stream is held up by a masonry dam or weir or by an earthen bank. In the former case there are innumerable examples in the small tanks that have been constructed in eastern countries where the slope of the ground is favourable. Of the latter there are also many examples on a very large scale : for instance, the Bhatgarh Reservoir in Bombay, the Periyar project in the Madras Presidency, and the more recent example of the Assuan Dam in Upper Egypt. These three works have already been referred to in Chap. II., which treats of the sections of dams. A few examples of the numerous storage reservoirs in the United States will be given later.

(2) In designing a reservoir, the first point requiring definite statistics is the maximum and minimum discharge to be expected from the catchment area, the former for the purpose of designing the works for the disposal of the surplus water which passes through the reservoir after filling the latter to its full capacity ; that is the required length of waste weir or by-wash or the discharging capacity of waste sluices, if such be adopted. The second determines the capacity of the reservoir, which naturally must be such that it will fill in ordinary years, and on this depends the area of land which it is capable of irrigating and the revenue which may be expected to accrue.

The subject of maximum discharge from catchment areas has already been investigated in Chap. V., par. 56; the minimum may be assumed as half the maximum, *i.e.*, with a maximum run off of $\frac{1}{4}$ of the rainfall, the minimum may be taken as $\frac{1}{8}$ or $\frac{1}{10}$ of the rainfall. There are, of course, many instances of small rain-fed tanks which naturally will not fill in a year of drought, though they may do excellent irrigation in ordinary years, so that in most cases the average rainfall has to be considered, not the absolute minimum. A tank, however, situated on a stream which never fails to run intermittently during the rainy season, even in a year of drought, is naturally more valuable than one dependent entirely on local rain, which may be very scanty in some years.

The ideal site for a reservoir dam is one across a narrow gorge in a stream or drainage line which above this point widens out in a long level and broad depression. The discharge of the catchment should be such that the tank will fill more than once during the season. According to the " Madras

Manual of Irrigation," the irrigating capacity of a tank thus supplied can be reckoned as 1¾ its actual capacity.

(3) The actual capacity of a tank is the cubic contents of the water impounded between full tank level (F.T.L.) or F.S.L. (full supply level), and that of the sill of the lowest irrigating sluice. One of the first points requiring attention, after selecting the site for a proposed reservoir, is the determination of its storage capacity up to different proposed levels of escape, *i.e.*, to F.T.L. For this purpose marks should be fixed at differences of level of about 5 feet or 10 feet apart, on any convenient short lines of section; the contours of these levels should then be marked out and surveyed all round the basin in order to obtain the perimeters and areas at each contour. From these the contents of each lamina can be calculated and the contents up to any contour. The same result can be obtained by a series of longitudinal and transverse sections taken up to the heights of the various contour levels. The former should be directed in conformity with the axis or axes of the figure of the basin, and transverse sections at right angles to them. Should a winding river channel or depression form part of the basin it is often more convenient and correct to estimate its contents independently and add it in afterwards. The above paragraph is taken almost verbatim from Jackson's "Hydraulic Manual," Chap. I., Sect. 4.

(4) The formulas in use for obtaining the contents or capacity from the horizontal contour areas are as follows:—

With two contour areas only, viz., $A_1$, $A_2$, at a common vertical distance apart $d$. The contents $= \frac{1}{3}d\ (A_1 + A_2 + \sqrt{A_1 A_2})$.

If there be three equidistant horizontal sections the contents $= \frac{1}{3}d\ (A_1 + 4A_2 + A_3)$.

If there be any even number ($n$) of equidistant horizontal sections $A_1$ $A_2$, etc., up to $An$ at a common distance $d$, the contents $= d\ (\frac{1}{2}\ A_1 + A_2 +$ etc. $+ An - 1 + \frac{1}{2} An)$.

The capacity of the reservoir being thus estimated, the amount of supply that can be expected annually from the catchment area can be obtained from the Tables given in Parts I. and II. of Table II., and Part I. Table III., Jackson's "Hydraulic Manual," where also some excellent practical examples of their application are given, or else the run off can be obtained from Tables given in VII. and VIII., Chap. V., of this work.

The irrigating capacity of tanks and duty of water impounded varies naturally with the amount of absorption and evaporation. In the Madras Presidency, where the system of irrigation from tanks and reservoirs is very extensive, the following is the quantity per irrigated acre required to be stored to bring the crop to perfection:—

| | Cubic feet. | Acre feet. |
|---|---|---|
| (1) Five months monsoon rice crop | 216,000 . . | 5 nearly. |
| (2) Cold weather rice crop . . | 175,000 . . | 4 " |
| (3) No 1, if stored for next year's use | 540,000 . . | 12·4 " |

This includes all loss from evaporation and absorption in the reservoir and the irrigating channels. This estimate, though rough, is useful as a guide. For cold weather crops other than rice, such as wheat, etc., the storage in acre feet will be about 2. The loss from evaporation varies with the climate of the country in which the reservoir is situated. In Upper Egypt the maximum is taken at ·39 inches per day, and that in India in the hottest and driest months is not believed to exceed ·4 inches, with ·2 inches per diem in the coldest month. The annual loss by absorption varies considerably according to the nature of the bed of the tank, and may be taken, according to "Irrigation in India," at one-half the yearly loss by evaporation.

From experiments made in tanks in Rajputana, which has a dry, hot climate, the average daily loss from both causes was—October to March, ·20; April to June, ·56; and July to September, ·41 inches per diem.

This is equivalent to a total annual loss in depth of 6·15 feet from evaporation and 3·62 from absorption, giving a total of 9·77 feet.

In Madras a continuous run of 1 cubic foot of water will irrigate 66⅔ acres of rice and double that amount of dry crops. In new tank projects the duty per cubic foot per second is sometimes taken as—rice 50, dry crops 100 acres, inclusive of losses from evaporation and absorption.

(**5**) Whether a masonry dam or earthen bank is adopted to impound the water, depends upon the local circumstances and comparative cost. The usual limit in height of earthen banks is 60 feet, but this has been greatly exceeded in some reservoirs in Bombay, where the Waghad reservoir embankment is 95 feet high, and another is projected of a maximum height of 110 feet. In England an embankment 125 feet high has been successfully constructed. With regard to masonry dams, there is no practical limit in height, although 200 feet may be considered exceptional. The Roosevelt Dam, Fig. 17a, is 260 feet high, the Shoshone arched dam, Fig. 22, Chap. II., is 308 feet high, and the Weir Croton Dam, Fig. 33, is 297 feet high.

The works connected with a tank or reservoir consist of (1) the embankment or dam, (2) the waste weir, by-wash or escape sluices for the disposal of surplus water, and (3) the outlet sluices for irrigation.

### (1) *Embankments.*

(**6**) The common practice in Europe is to construct a puddle core in the centre of an embankment, the remainder of the material being composed of earth, gravel and stones; in this case the imperviousness of the dam to leakage is entirely dependent on the puddle core.

Where suitable earth is not obtainable to form the mass of the embankment, the adoption of a puddle core is obligatory, but where good soil is available the puddle core for embankments of, say, 50 feet in height is not necessary. In India the use of a puddle core is generally limited to very large reservoirs, but even in the case of high banks it is often entirely dispensed with. A thoroughly consolidated homogeneous embankment of earth is undoubtedly preferable to one of infirm material rendered watertight by a puddle wall. The large Waghad tank embankment, a section of which is shown in Fig. 1, was constructed with rammed layers of moist earth without a puddle core.

(7) In the Madras system of construction, as shown in the diagrams in the "Madras Irrigation Manual," the puddle core appears to be used even in quite small embankments, examples of which are given in Figs. 2 and 3. Unless clay soil is not easily obtainable near the site of the embankment, this adoption of a puddle core for these small depths of water is, as already noted, quite unnecessary. In forming an embankment without a puddle core, it is essential that the earth be at least damp and consolidated in layers by heavy rollers. The whole mass can be made of wet earth, *i.e.*, practically puddled, if water is available. The method of procedure is as follows:—The bottom 10 feet of the base of the bank should first be thrown up, and a temporary cut made in the solid ground or rock at a lower level to pass off waste water, or else the masonry outlet culverts can be first built and adapted for this purpose. The work should then lie in abeyance until water has collected in the basin behind the bank. The surface of the banks should then be divided into shallow basins about 12 inches deep by narrow partition walls of earth. Into these enclosures the water should be pumped or baled up from the reservoir. As soon as a series of these shallow basins are full of water, the earth is thrown in to fill them up level with the top of the partition walls, after which another series of chequers are formed on top and again watered. While one part is being filled up, another is being watered or chequered, so that there is no intermission in the earth carrying.

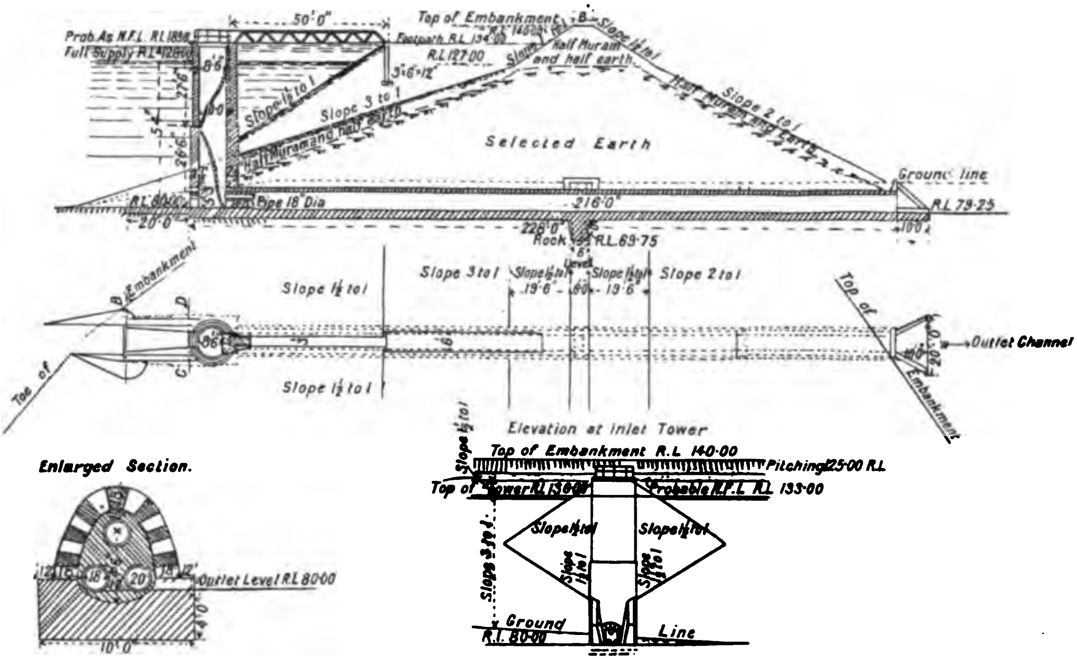


FIG. 1.—Waghad Tank Embankment.

When the embankment is thus raised, the level of the bed of the escape cut can likewise be raised either by partially filling it up or cutting a new channel at a higher level, so as to allow the water to rise to a further height behind the bank. By the means thus described, each layer of earth is thoroughly soaked and clods dissolved, so that no ramming or clod breaking is requisite, and the new layer is further consolidated by having 6 or 9 inches of water laid over it, the result being that the whole bank is composed of wet earth devoid of air spaces, which are inseparable if dry earth is used, no matter how much it may be consolidated by rolling or ramming. Consequently when the tank fills, there can be no settlement whatever of the embankment. If during this process the bank shows signs of supersaturation by bulging,

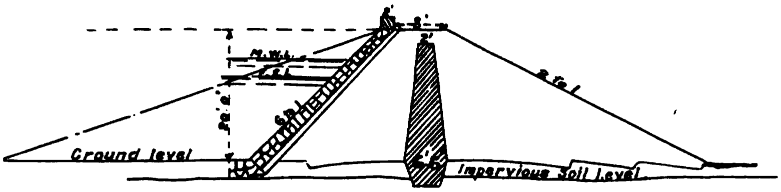
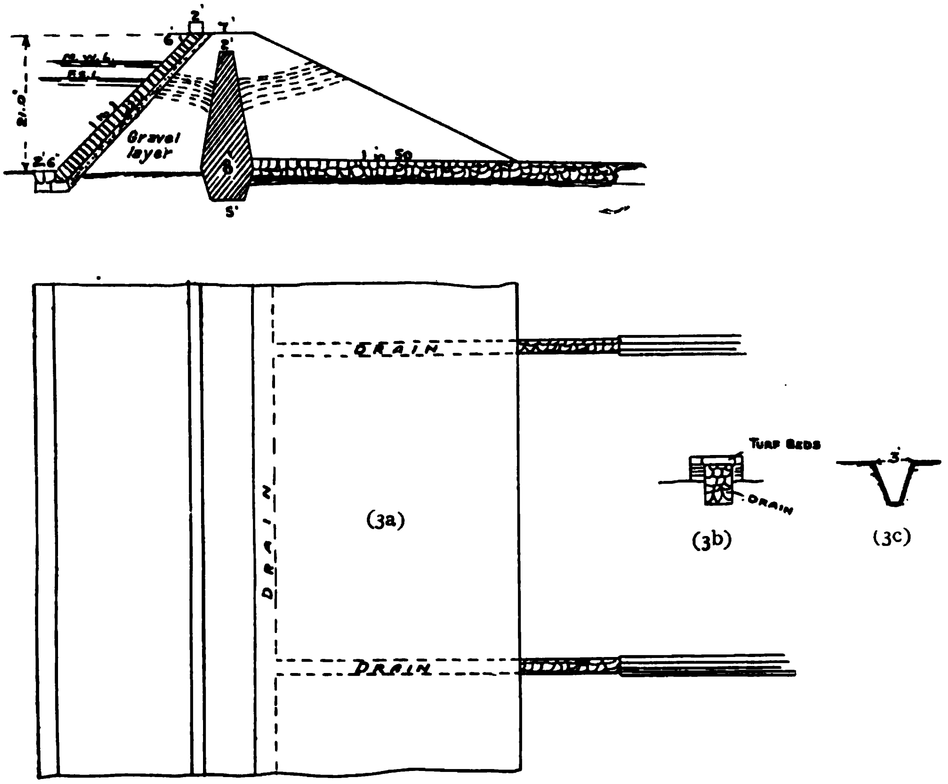


FIG. 2.



(3a)

(3b)

(3c)

FIGS. 3, 3a, 3b, 3c.

the work should be suspended at this part for a day or two till matters adjust themselves, and the layer of water subsequently reduced in depth.

The author has constructed several embankments on this system, and not the least sign of settlement ever appeared in the bank thus formed when fully tested. This system is probably only applicable where coolie labour is employed and earth carried on the head in baskets, when it can be deposited wherever required.

In many cases where stiff clay is the material, it is customary to mix a certain proportion of fine gravel with the clay, as gravelly soil is naturally far more watertight than pure clay, unless the latter is kept continually moist. This can be done by arranging a certain proportion of the carriers to convey gravel, so that the mixing takes place automatically. In the United States the universal practice is to construct the core walls of masonry, puddle never being used for this purpose. The embankments there are thrown up in layers of damped earth and rolled by heavy rollers.

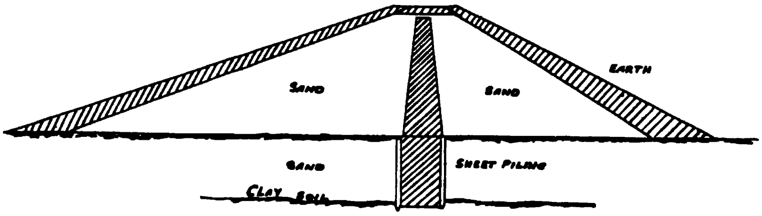


FIG. 4.

(8) It often happens that an embankment has to be thrown across the sandy bed of a stream, as in the case of the diversion of a stream down a cut at a higher level than its bed. In such cases, up to a depth of, say, 30 feet, it will not be necessary to excavate the sand substratum or even put a puddle wall down, as although leakage will doubtless occur through the sandy bed below the embankment it will be quite innocuous, the weight of the embankment causing frictional stability exactly in the same way as the drop wall and floor in a river weir.

(9) The weight or value of $Ap$, in the case of an embankment, is always considerably in excess of $CH$, or of the head of water multiplied by a suitable coefficient (*vide* pars. 19—21, Chap. VI.). The ratio of the length of creep ($l$), provided by the base width of the bank, to the head is, however, deficient. In the Jeypore Dam, the value of $Ap$ is four times in excess, while that of $l$ is one-half of requirements, the latter being $8\,H$ only. In order to provide this necessary length of creep, a puddle floor should be carried up stream of the inner toe of the embankment for a length sufficient to make $l$ not less than 10 or $12\,H$; its depth need be only just sufficient to make the creep effective (say 2 or 3 feet), the weight required for frictional stability being already more than is necessary. Successful practice in the construction of many earthen dams on sand goes to prove the fact that if $Ap$ is much in excess of the requirements of frictional stability, that of $l$ can be safely curtailed to a maximum value of $10\,H$. Sheet piling has a powerful effect in increasing the value of $l$ if required.

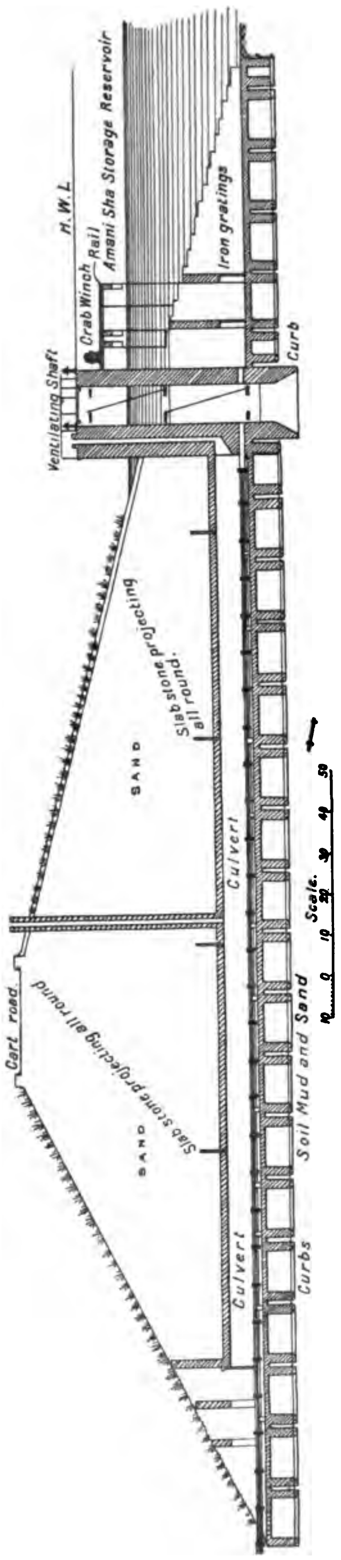


FIG. 5.—Jeypore Waterworks. Sand Dam.

**(10)** When the layer of sand below an embankment is shallow, overlying a clay substratum, leakage which may be dangerous is liable to occur at the junction of the two materials; in such event sheet piling is a necessary adjunct, or else a puddle wall enclosed between two rows of sheet piling. This construction is exemplified in Fig. 4. Here the bank is shown made of sand covered externally with a layer of clay, and the puddle core is taken right through into the solid clay. This construction is suitable for a reservoir embankment where silt deposit is not to be reckoned on.

**(11)** A very instructive example of the capabilities of embankments made of pure sand is given in Fig. 5, of the embankment of the Jeypore Waterworks, taken from the "Min. Pro. Inst. C.E.," Vol. CXV. In this case the embankment is not only made entirely of sand but also lies on a foundation of sand of great depth. The H.W.L. is 44 feet above the floor of the sluice, which is at the bed level of the original stream. The free board or margin between H.W.L. and crest of bank is greater than usual, being 15 feet, *i.e.*, one-third depth of water, and the crest width is 32 feet.

**(12)** The usual dimensions of a clay embankment, where a wide roadway is not required, would be, top width 15 to 20 feet and free board $\frac{1}{4}$ to $\frac{1}{5}$ depth of water, with a minimum of 3 feet and a maximum of 6 feet and face slope three to one, back slope two to one, whereas in this case the face slope is four to one. The pipe culvert is founded on blocks of brickwork sunk 8 feet below floor and filled with concrete; the inlet tower is founded in like manner. The leakage is said to be insignificant, the silt deposit on the bed of the tank having stanched the sand.

**(13)** Fig. 6 shows the founding of a puddle wall on concrete which continues the impermeable core down to solid rock. In some embankments the puddle trench has to be carried through fissured or inferior rock, in this way to even 100 feet in depth. Rock, unless of the nature of solid bed rock, is not so secure a foundation as sand, as pressure might disintegrate fissured rock and blow it out, causing the destruction of the embankment, whereas with sand the superimposed weight presses the particles together and prevents failure by the frictional stability thus induced. Some leakage is sure to occur in the sand substratum, but it will generally be harmless.

(**14**) With regard to the thickness of puddle walls, the following extract from "Waterworks Engineering," a recent standard work on the subject, will give a safe general rule:—"For general guidance it may be regarded as a safe rule to make the thickness of the puddle wall at the base of the embankment equal to one-third the depth of the impounded water in the reservoir, battering both sides at such an angle as will give a common thickness of 6 feet of puddle at the top of the wall." This dimension would only apply to a very large embankment. "It is unnecessary to carry the puddle wall above the highest wave level of the reservoir, and it is undesirable for it to have anywhere a less covering than 3 to 4 feet of ordinary earth to protect it from the co-operative influences of sun and wind. The puddle wall must be carried down through the subsoil to form a watertight joint with the impervious base of the reservoir. It is laid in a trench excavated in the ground the sides of which, frequently battered equally but always reversely to the sides of the puddle wall in the embankment, form the requisite lateral support to the material. In practice, the thickness of this wall at the base is seldom found to be less than half the thickness possessed by it at the ground level." The puddle core is sometimes formed of an intimate mixture of sand, gravel and clay. In the United States peat has been used for the same purpose, but in that country the core is almost invariably made throughout of rubble masonry or concrete or steel plates enclosed in concrete.

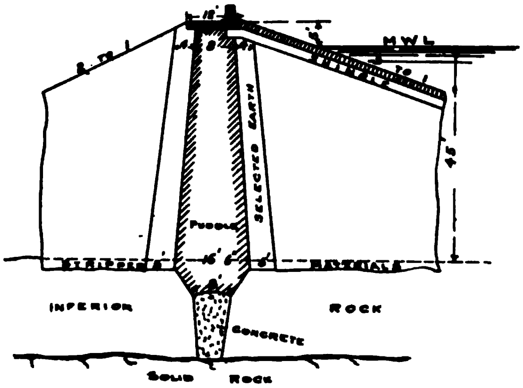


FIG. 6.

The Madras rule for the thickness of puddle walls is to make them 2 feet thick at top with a side batter of one in eight or one in six; the base width would then become $\frac{d}{4} + 2$ or $\frac{d}{3} + 2$; not very far from the rule quoted in last paragraph of $\frac{d}{3}$; the side batter is, however, much greater and the top thinner. It should be borne in mind, however, that the earth when carried by coolie labour is far more evenly distributed and much better consolidated than when tipped by trucks in the usual European method, hence a thinner section of puddle wall can be adopted.

(**15**) We have already seen (par. 12) that the usual safe side slopes to embankments are three to one on the water side with two to one rear slope. As regards top width, this should not be less than 8 feet in a small tank impounding 10 or 15 feet of water, increasing to 15 or 20 feet in a high embankment. It is evident that the top width should, as in the case of a masonry dam, be some function of the depth of M.W.L., and the general rule adopted in Madras that the width of an embankment at M.W.L. should not be less than the maximum depth appears worthy of general use. Thus with 50 feet depth and 6 feet free board and three to one fore and two to one back slopes, the thickness of crest will become $50 - (5 \times 6) = 20$ feet. But this rule would not be operative for depths much below 40 feet. A direct rule for the top width, irrespective of the height of free board, of $2\sqrt{d} + 2$ would seem to be in agreement with ordinary practice. Thus with M.W.L. at 16 feet depth the crest width will be $2 \times 4 + 2 = 10$ feet; at 25 feet depth $(2 \times 5) + 2 = 12$ feet; at 40 feet depth $(2 \times 6) + 2 = 14$ feet, and at 50 feet depth $(2 \times 7) + 2 = 16$ feet.

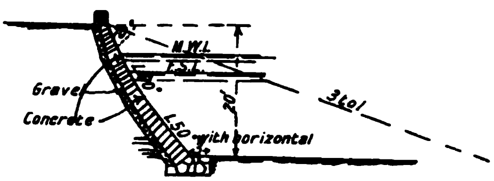


FIG. 7.

The Madras rule for top width is $\frac{d}{4}$ with a minimum of 8 feet in long tank embankments with varying depths.

(**16**) The height of the free board above M.W.L. is seldom less than 10 feet in large embankments, or than 3 feet in small ones; the free board is subject to the conditions of wave action *vide* par. 18.

The section of the Waghad Embankment in Fig. 1, with a top width of only 6 feet, is not considered a good one by Col. Mullins, by whom the group of Bombay tanks are critically reviewed in the Appendix to the "Madras Irrigation Manual."

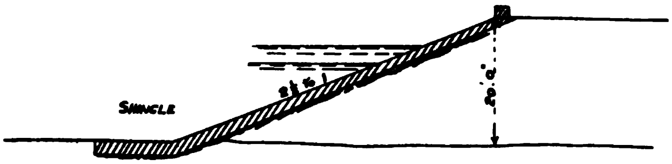


FIG. 8.

(**17**) Most embankments require protection by stone pitching where exposed to the wash of waves. In such cases, where the depth is moderate the fore slope can be reduced from three to one to one to one for a certain length where a masonry outlet occurs; examples of this are given in Figs. 1, 2, 3 and 7, the latter with curved fore slope. Fig. 8 represents a fore slope at two-and-a-half to one covered with shingle, which will stand at that angle, and if obtainable forms an effective protection from wash. On parts of a reservoir bank, which from their position are not exposed to wave action, the outermost layer on the bank can be composed of gravel. Reed grass also forms an excellent protection, and in India it is pulled fresh, formed into a mat roll and thus spread over the surface and covered with a sprinkling of earth when it readily takes root and will grow in 2 or 3 feet of water.

The pitching as well as the puddle wall should extend at least as far as the wave action will reach.

**(18)** The formula for the height of the waves in feet above M.W.L. is $1{\cdot}5\ \sqrt{F} + (2{\cdot}5 - \sqrt[4]{F})$ where $F$ is the fetch or longest line of exposure of water surface to wind, expressed in statute miles.

Thus, if $F = 4$ miles the height will be

$$1{\cdot}5 \times 2 + (2{\cdot}5 - 1{\cdot}4) = 3 + 1{\cdot}1 = 4{\cdot}1 \text{ feet.}$$

With $F = 10$ miles the height will be $1{\cdot}5 \times 3{\cdot}1 + (2{\cdot}5 - 1{\cdot}7) = 5\frac{1}{2}$ feet.

**(19)** Embankments, where the soil is unfavourable, are often provided with drains longitudinal behind the puddle wall with transverse ones at intervals. These are trenches filled with loose stone in order to carry off any leakage and prevent the rear portion being supersaturated, thus preventing slips. An example of this is shown in Figs. 3 and 3a, *ante*.

### *Dams.*

**(20)** A masonry dam has this advantage over an earthen embankment in that it can act in whole or in part as a waste weir, as for instance the Vrynwy Weir, 60 feet high, which has an overflow; the Coolgardie Weir, 120 feet high, and the Bhatgarh overflow weir; or else waste or supply sluices can be built in the body of the wall, as has been done in the case of the Bhatgarh and Assuan Dams. This form of waste sluice, now that so successful an example on a very large scale has been constructed at Assuan, is bound in many cases to supersede the commonly adopted overflow waste weir distinct from the dam.

**(21)** The design of the section of dams and the rules governing the shape of the profile have been fully treated in Chap. II.; it only remains to mention the following points: In a long dam where no roadway is required on the crest which necessitates a uniform crest width, this latter should be proportioned to the depth, *i.e.*, as stated in par. 16, Chap. II., where the depth $H$ is at or above 50 feet, the top width will be $\sqrt{H}$. Below 50 feet the top width will be $\frac{1}{2}\left(\frac{H}{\sqrt{\rho}} - \frac{H}{10}\right)$. The bottom width will likewise vary as $\frac{H}{\sqrt{\rho}}$. Thus the dam on plan will have a somewhat curved outline diminishing towards the ends.

If the dam is such that surplus water falls over it, it becomes a weir and will have to be designed in accordance with the rules laid down in Chap. III.

(**22**) The requisite height of a stop dam (not weir) above the highest water level in the reservoir or M.W.L. is found by the application of the formula given in par. 18. In calculating the base width this higher level is taken as the reservoir level (not the actual M.W.L.), as was done when dealing with calculations for the Assuan Dam in Chap. II.

(**23**) The material of which a dam or weir is constructed is usually either rubble masonry or concrete. The following remarks on this subject, from "Waterworks Engineering," are well worthy of reproduction :—

"The construction of the vast masses of masonry or concrete of which dams are formed, is an art that requires the exercise of judgment in the selection of the materials used, close attention to their preparation, and watchfulness during the process of building. The question as to whether concrete or rubble masonry is in any given case the preferable building material must largely depend upon the character of the rock available. The great tensile strength of Portland cement causes it to be in high favour as a matrix, but it is not entirely unexceptionable. Where a dam abuts, as is frequently the case, upon steep hill sides, the rapid variation in height tends to produce an unequal settlement in the masonry or concrete as the structure is gradually raised. This settlement is resisted by the shear of the material transmitted from the comparatively unyielding rock abutments. It is not improbable that action of this kind has produced some of the fractures that have too often occurred in such dams. If this be so, it points to the prudence of building in a slow-setting hydraulic lime, thereby permitting free settlement of the mass before it sets rigidly. The rapid settlement during construction of large masses of concrete must give rise to internal shearing stress of indeterminate amount. In concrete work, such settlement is much reduced by the introduction of large blocks of stone and boulders into the work, a practice that possesses the advantages of adding to the coherence of the entire work and generally of effecting some economy in its cost."

(**24**) To which remarks might be added that the use of long header stones in rubble masonry laid cross ways, which is often specified, tends to facilitate rupture of the wall. With a battering face, each face course overlaps the preceding one, thus automatically forming a bond between the face work and the interior filling, so that long bond stones are not only unnecessary to preserve transverse bond, but they have the disadvantage of breaking the longitudinal bond. A wall under pressure from the rear has no tendency to split longitudinally as a badly bonded wall of a building might have, but owing to unequal settlement the tendency is for transverse cracks to be formed. Thus, if used at all, which is to be deprecated, bond stones should lie longitudinally, not transversely. The author considers that the use of special bond stones in rubble masonry is a mistake as tending to destroy the homogeneity of the mass.

(**25**) In the Bhatgarh Dam, a section of which is given in Fig. 18, Chap. II., the interior was formed of concrete with a plentiful inter-sprinkling of large stones which were imbedded in the concrete. This raised the specific gravity of the mass considerably above what it would have been without the admixture of heavy stone blocks, the actual weight rising to 150 or 160 lbs. per cubic foot.

The Periyar Dam (Fig. 15, Chap. II.) is also formed of concrete.

It has already been stated that the construction of sluices in the body of a dam, as has been effected in the Assuan and Bhatgarh Dams, would be a more economical procedure than that of a separate waste weir, unless natural conditions specially favoured the cutting of an inexpensive by-wash.

### *Disposal of Surplus Water.*

(**26**) Where the main dam of a reservoir does not itself act as a waste weir by allowing surplus water to pass over it, or through it (if waste sluices are adopted) a separate waste weir or by-wash has to be constructed. Such is naturally also the case where the dam is of earth. The safe disposal of surplus water passing through an irrigation tank which is often the receptacle of a large stream is a most important matter, and the ruin of most old native irrigation tanks is generally due to proper means of escape of surplus water not having been arranged for. For designing such a work the maximum possible inflow into the tank under the assumption that the latter is already full has to be ascertained, and the waste weir built of such a length as to be able to discharge this quantity or rate of inflow at a defined depth of film of water passing over the crest. This depth (*d*) is very often 3 feet, but may be made anything in reason provided that the expense of raising the embankment all through to this extent does not exceed the cost of providing a longer waste weir with a less value of *d*.

(**27**) It is evident that the crest level of the waste weir is that of full supply level or F.S.L. of the reservoir, while the maximum tank level or M.T.L. (to use the Madras phraseology) will be F.S.L. + *d*. Now the level of the crest of the embankment or dam is naturally dependent on the latter, not the former, so that the adoption of a waste weir as a means of disposal of surplus water involves the raising of the whole embankment to an extent equal to *d*, the greatest allowable depth of film passing the weir. This forms the great drawback to the adoption of waste weirs, which otherwise are an excellent provision for escapes, being self-acting.

(**28**) In order to reduce the difference between F.S.L. and M.F.L. to a minimum, if not to abolish it altogether, several courses are open. Firstly: The weir could be provided with collapsible shutters, as are commonly employed on river weirs, and described in Chap. VI. These are balanced at the centre of pressure, and fall automatically when overtopped. They will have to be raised by hand, which could be effected on the lowering of the water level in the reservoir by men hooking them up into position from a staging erected over the weir. This method is doubtless economical and safe, but requires very judicious regulation to prevent loss of storage water, and is practically limited to depth of 3 or 4 feet. Secondly: Automatic drop shutters could be adopted, as exemplified in the Bhatgarh waste weir (Fig. 9). This is the only system purely automatic in action, the gates being lowered on any increase of level in the water of the reservoir, while on a fall of the same below F.S.L. they rise and close the openings. Thirdly: Sluice bridge openings could be adopted, similar to what was suggested for the Koshesha Escape in Chap. X., with collapsible upper gates and lower draw gates, or only with the former. Fourthly: An overfall of any kind could be abandoned and the regulation effected by low level sluices built in the body of a closed dam, as exemplified in the Bhatgarh Dam and the Assuan Dam (Chap. II.). Fifthly: The cylinder rolling on crest, as shown in Fig. 35.

(29) The design of the self-acting shutter and gates is illustrated in Fig. 9, which is taken from "Irrigation Works in India."

The principle is that of a counterweight, placed in a deep cistern constructed under each pier, which counterweight is connected with the gate by a system of chains and pulleys. This gate is hung vertically, and runs on anti-friction rollers. An inclined channel through the pier connects an opening which is placed at F.S.L. with the base of the cistern; when water rises above F.S.L. it obtains access to the cistern and raises the counterweight and lowers the gate; when the level in the reservoir falls below F.S.L. the supply of water to the cistern is cut off and the counterweight falls, bringing the gate up into position again. There is

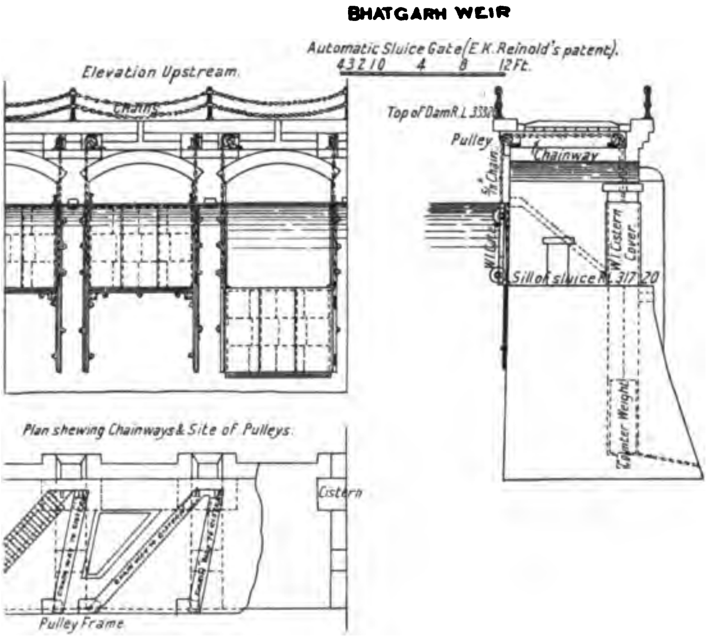


FIG. 9.

a small outlet at the base of the cistern. These gates are said to have given satisfaction experimentally, and were stated to be in course of erection on the Bhatgarh weir. It would be interesting to know how they succeed in actual use. In want of any definite information on the subject, in the author's opinion the success of this system in regulating the outflow is but problematical, for the following reasons. Firstly: The force of the current passing over the gate, and the weight of water, would tend to keep the gate down, once lowered, and destroy the equipoise. Secondly: During storms, water would obtain access to the opening to the cistern and cause the gates to fall when not required. Further, the arrangement involves considerable expense, so that the advisability of its general adoption is open to doubt.

**(30)** A section of the Goulburn Weir at the head of the canal system of that name in Australia is given in Fig. 10. This quite recent work is a drop gate weir. Reinforced concrete piers 2 feet wide are erected on the crest 20 feet apart, and these spaces are closed by drop gates 20 feet long by 10 feet deep. The gates, which are manipulated by rack and pinion gearing, are lowered into recesses constructed below the weir crest. Owing to the extreme thinness of the piers and the weakening of the section by the gate recess below, the weir has to be strengthened by extensive steel reinforcement right down to the level of the base of the recess, and the framework of the piers is bolted down to this lower reinforcement.

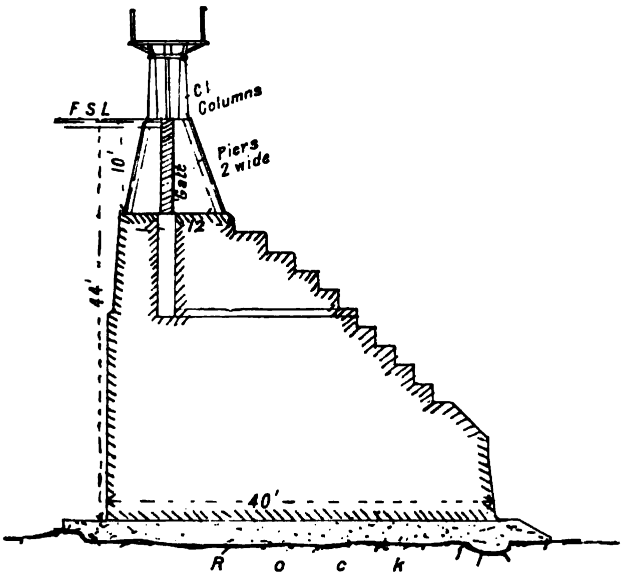


FIG. 10.—Section of Goulburn Canal Weir.

**(31)** This arrangement, which clearly involves considerable expense with no corresponding advantage, cannot be commended. A better arrangement undoubtedly would be to substitute solid wide masonry piers, arched over in the usual Indian style. The drop gates can be disposed of by being arranged to slide down the inner face of the weir, and when drawn up will run in roller paths up the face of the piers. The extra thickness given to the piers will necessitate lengthening the weir, but any increase of masonry in this respect can be economised in the base thickness, which is unnecessarily great.

These suggested improvements are embodied in Fig. 11.

The base thickness of the weir is reduced from 44 feet to 30 feet, or $\frac{H + \cdot 6d}{\sqrt{p}}$, which is ample for purposes of stability, and the weir crest proper is increased to 15 feet width. A batter of one in five is given to the rear face. This will facilitate the manipulation of the roller gates, each of which will be lifted by chains passing up the pier grooves at either extremity, the chains being brought on to either a travelling, or two fixed and connected winches, resting on the arched platform.

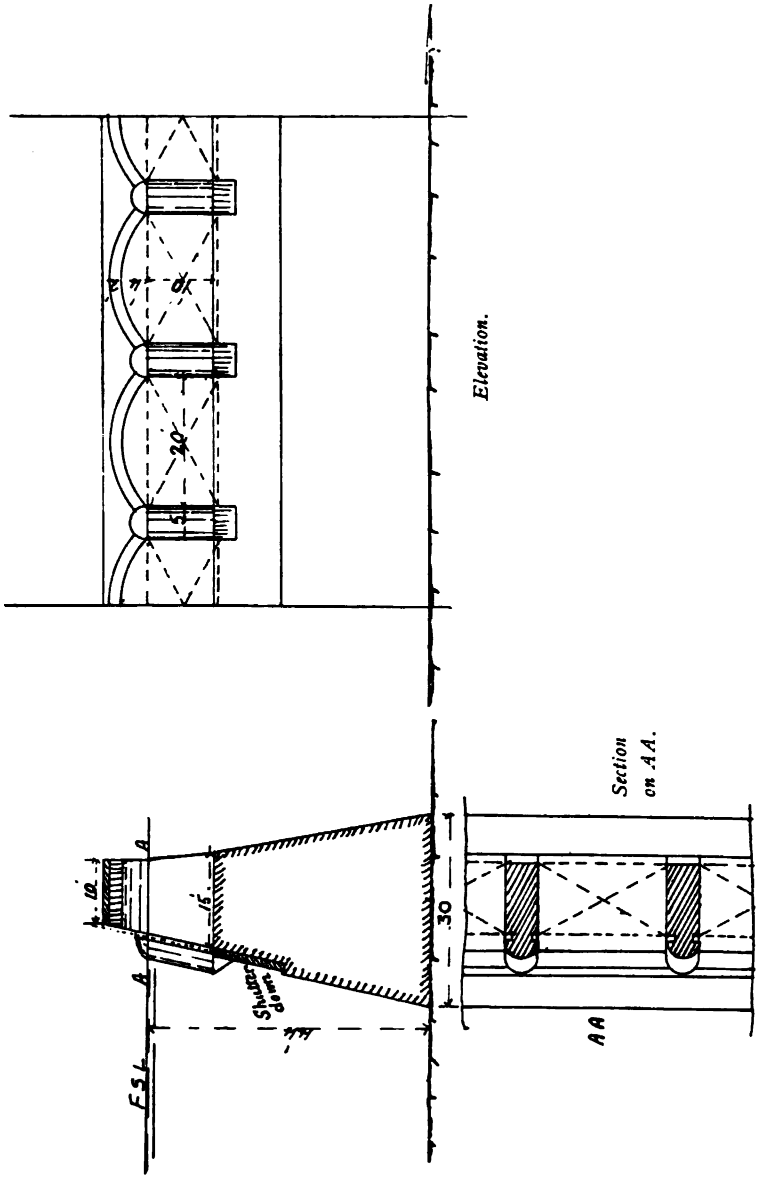


FIG. 11.—Alternative Design for Goulburn River Weir.

This is undoubtedly a simpler and cheaper design than that shown in

Fig. 10. The excessive base thickness given is due to the old exploded prejudice against allowing a free overfall, which still shows remarkable vitality in irrigation designs of the Western and Southern Hemispheres.

**(32)** In all probability the fourth system, mentioned in par. 28, viz., of sluices similar to those in the Assuan Dam, provided with Stoney's anti-friction roller gates, which can easily be lifted by one man, would be the best form of escape for surplus water in large works. For instance the Bhatgarh waste weir is built over the rocky bed of what is probably an old water course, and the level of M.F.L. is from 30 to 20 feet above this bed. There is therefore ample space vertically for working sluice gates.

The Bhatgarh waste weir, as at present constituted (*vide* Fig. 9) has forty-five openings of 10 feet span; the effective width of each is $10 - \cdot 2\frac{d}{4} = 9\cdot 4$ feet, or say $9\frac{1}{2}$ feet; the discharge of each opening will be from Table II., Chap. V., with $d = 8$, 75·4 cubic feet per foot run, or $75\cdot 4 \times 9\cdot 5 = 716\cdot 3$ cubic feet per opening, giving a total flood discharge of $45 \times 716\cdot 3 = 32{,}233$ cubic feet per second. Supposing a closed dam to be substituted for the present weir, provided with sluices placed with their top sills 8 feet below what is the present weir crest level, their sectional area will be 8 feet × 10 feet, and on the supposition that they are just submerged, the discharge of each will be, by Formula (3), Chap. V., par. (3).—

$$Q = c \times A \times 8\cdot 025\ \sqrt{H}$$

In this case $H = 8$, $A = 80$. Taking $c$ as ·84, the expression becomes $\cdot 84 \times 80 \times 8\cdot 025 \times 2\cdot 82 = 1{,}521$ cubic feet discharge for each sluice. The number of sluices would then be $\frac{32{,}233}{1{,}521} = 21$.

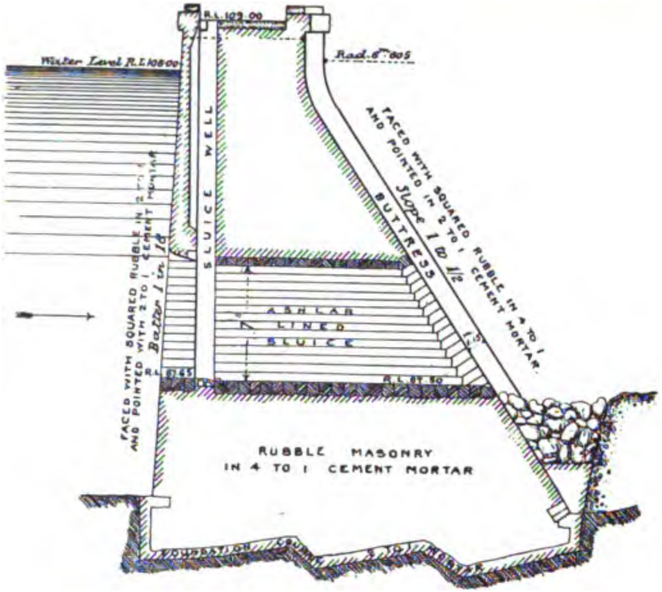
As will be seen from the longitudinal section (Fig. 18c Chap. II.), most of the sluices could be placed at a still lower level, and the discharge required could thus be obtained by no more than fifteen or sixteen sluice openings; the dam could then be constructed in section similar to the Assuan Dam.

Fig. 12 is a section of the Assuan Dam, as it stands at present, through the lowest tier of sluice openings. The dam as raised to the originally contemplated level of R.L. 117 has already been illustrated in Fig. 19, Chap. II. These sluices are 2 metres wide and 7 metres high, and the piers are 5 metres in width, with occasional abutment piers.

The lifting arrangement with Stoney's anti-friction rollers is illustrated in Fig. 12, Chap. XIV.

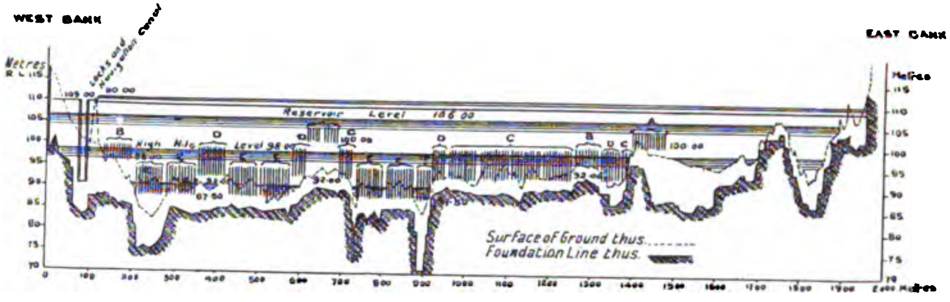
An elevation of the dam showing the position of the various tiers of sluices is given in Fig. 12a, and a plan over all in Fig. 12b.

**(33)** The sluices in a dam could be placed at different levels in accordance with that of the rocky bed on which the dam is built, or else grouped together at, say, 20 feet intervals in the centre of the dam. The section of the dam can be increased by buttresses projecting beyond the normal back slope, in the intervals between the sluices, as at Assuan, thus



Water Level R.L. 108·00
R.L. 109·00
Rad. 6·805
SLUICE WELL
FACED WITH SQUARED RUBBLE IN 4 TO 1 AND POINTED IN 2 TO 1 CEMENT MORTAR.
Slope 1 to 1/2
BUTTRESS
FACED WITH SQUARED RUBBLE IN 2 TO 1 AND POINTED WITH 2 TO 1 CEMENT MORTAR
Batter 1 in 14
ASHLAR LINED SLUICE
R.L. 87·45
R.L. 87·50
RUBBLE MASONRY IN 4 TO 1 CEMENT MORTAR

SECTION OF PIERCED DAM.

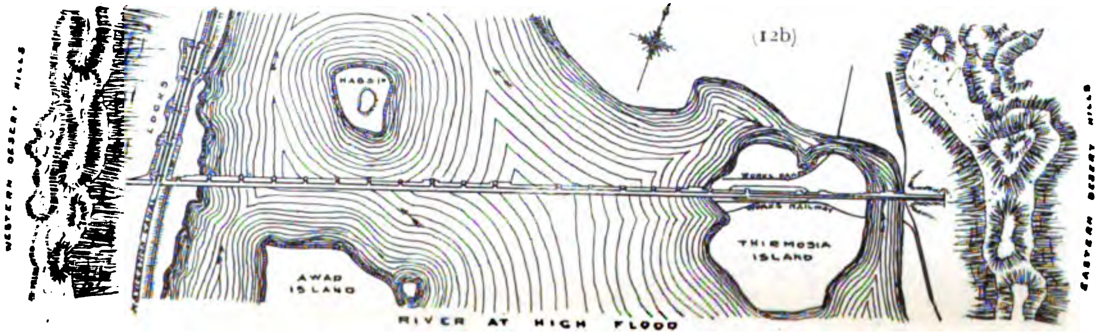


ELEVATION OF SOUTH SIDE OF ASSUAN DAM

A. 22 sluices at R.L. 100
B. 18 ………… 96·0
C 50 ………… 92·0 (without rollers).

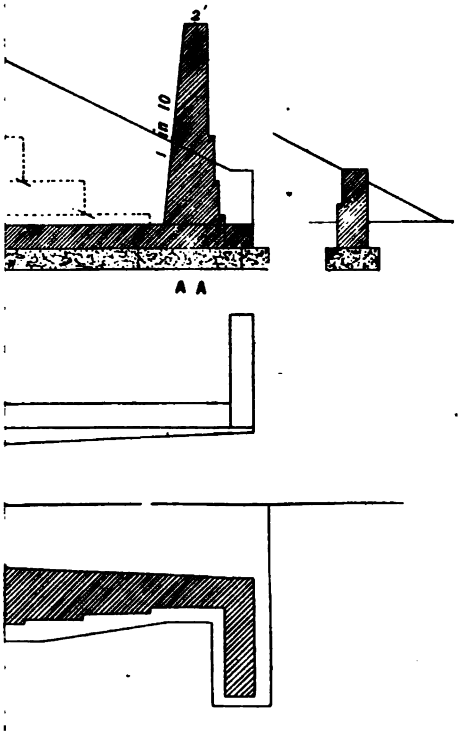
D. 25 sluices at R.L. 92·0 (with rollers)
E. 65 ………… 87 5

(12a)



(12b)

FIGS. 12, 12a, 12b.—Assuan Dam (crest at 109·0).



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affording compensation for the voids in the sluice channels. These buttresses need not be carried much above the top of the sluices. The rear panel wall forming the sluice well, as in the Assuan dam, could be adopted in this case, though not absolutely necessary, as the gates can well be exposed outside.

(**34**) Sluice gates such as these can be raised in a few minutes by a travelling winch, or by a fixed gear attached to each with removable handle bar. On the occurrence of floods in a large reservoir the water takes some time to rise, so that there is no danger from any sudden increase in the depth. Besides which, apparatus could easily be fitted that by means of a float in the tank could give the alarm to the watchman on any rise at night time by sounding an electric bell or discharging a detonator.

One great advantage possessed by a dam provided with sluices over an overfall weir is that the sluices will scour out the deposit in the tank bed, a function which an overfall weir naturally cannot effect.

(**35**) The third system (*vide* Fig. 13) consists of an open arch bridge way, within which are fixed the turn-over pivot gates described in par. 25, Chap. X., and illustrated by the sketch in Fig. 10, Chap. X. These gates can be adjusted to turn over automatically when topped, or by means of a catch in the winch or windlass that revolves with the chains, can be held upright in any position and released by a single blow when required. They can also be raised to any angle by revolving the power winch. One 10 feet square opening combined with a fall will discharge nearly 900 cubic feet per second, so that sluice bridges of this description, which are inexpensive and easily worked, are well suited for openings in embankments to pass surplus water. An ordinary waste weir with 3 feet depth of film will discharge (*vide* Table II.) only 17·3 cubic feet per foot run; and to dispose of 1,000 cubic feet per second would have to be nearly 60 feet in length, so that the open sluice way of moderate depth combined with a fall would undoubtedly be much more economical. As before stated, some of the gates can be made to open automatically, while others can be retained in position until required. When the depth of water is considerable and a long masonry dam is necessary, with a roadway built over it, the construction could be similar to the Bhatgarh Weir, consisting of piers and arches built over the weir crest, which can be placed 10 or more feet below F.S.L., the openings being 10 feet or 12 feet square. This would apply to a shorter weir with less surplus to dispose of than 30,000 cubic feet per second; otherwise low level sluices in a dam, as previously suggested, would be more suitable.

(**36**) A short description will now be given of the example illustrated in Fig. 13 of an open sluice for supply or waste purposes fitted with the author's revolving gate. The gate is supposed to be of wood, and the drawing is little better than a sketch; it is, however, quite sufficient for the purposes of illustration. The chains at each end pass round drums which are connected by a horizontal shaft along the centre of the bridge. On the centre of shaft will be keyed a cog wheel which is engaged by a pinion mounted on a frame and turned by a handle; this pinion can be easily disengaged; the shaft is then free to revolve with the gate. The chains passing over the end drums can be tightened or slacked as required by operating a screw which will raise or lower the standards carrying the drums.

Smooth cast iron plates projecting slightly are let into the wall in the line of the gate when upright. The space left between the ends of the gate and these plates, about half an inch, will be closed and kept watertight by stanching rods of round iron attached to the gate. These are shown in the end elevations.

(**37**) An example of the design of a tank waste weir is given in Fig. 14. The assumed data governing the design are—

*d*, or depth of film = 3 feet.
*D* the depth above floor in escape channel = 6 feet.
Then $r^{\circ} = \frac{d}{D} = \cdot 5$.
Fall in surface, 20 feet.

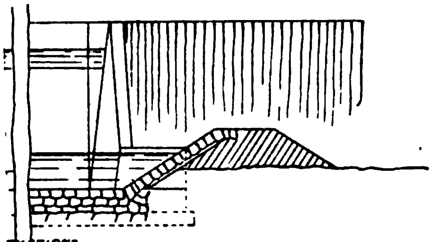
The surface level is assumed to be 3 feet above floor level, although this hardly affects the design. The free board is taken as 5 feet. The height of the abutments will then be $20 + 3 + 5 = 28$ feet. The section of the weir will be of the full, not the "hybrid" type, the use of which, except in cases where the weir wall itself is in cutting and the puddle at back can be securely filled in between the back of the wall and the face of the cutting under careful supervision, is to be deprecated as unsafe.

According to par. 17, Chap. III., the crest width will be $\sqrt{H_1}$ or $23 = 4{\cdot}8$ feet.

The base width, according to Formula (5), par. 21, Chap. III., will be $\frac{H_1}{\sqrt{\rho}}\left(D \text{ being less than } \frac{H}{2}\right)$ or $23 \times \frac{2}{3} = 15\frac{1}{3}$ feet (made 15 feet 6 inches). The profile of the weir wall is similar to that given in Fig. 7, Chap. III.

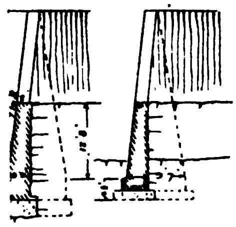
The arrangement of wings adopted is a simple one. The embankment slopes are three to one rear and two to one fore. The approach wings will be inclined at one in ten to the axis of the work and will be of the sloping crest variety, terminating at the intersection of the slope with the natural surface. The face wings will similarly slope down, their length being determined by that of the floor, and ending in a direct return dwarf wall. These will also be inclined on plan at one in ten. The wings will have a face batter of one in eight. Consequently their base thickness at starting point will be, according to Table I., Chap. I., $= \cdot 4H - 1\frac{1}{2}$. This at the starting point at junction with the abutment will be $(\cdot 4 \times 28) - 1\frac{1}{2} = 9{\cdot}7$ feet. The back batter will disappear when the height of the wall is lowered to 12·8. This point will be 45 feet 6 inches from the abutment on the rear walls and 30 feet 6 inches on the face walls. The base width here will be 3·9 feet. As usual, the top width is taken at 2 feet.

The length of floor, which is measured from the toe of the weir wall, is $2H_1 = 46$ feet (*vide* par. 20, Chap. IV.). Its thickness will be $\sqrt{H_1} + 1 = 5{\cdot}8$



EVATIONS

Down Stream



Sections B B

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(say 6 feet). The abutment, being mainly supported by the weir wall, is made 5 feet thick, with vertical face and back. The battered backs of the wings overlap the sides of the abutments 1 foot.

The escape channel gradually widens at one in ten, and is shown embanked below the work, which would necessarily be the case if the natural ground were as assumed, at only 3 feet above flood level.

In tank falls the escape channel is generally made, if possible, wider than the weir, to reduce the depth *D* and consequent velocity of current. In such cases the depth in the escape channel is dependent not only on its bed slope, but its area. Examples of the type of fall most suitable for a waste weir in cutting are given in Fig. 24, Chap. I., and Fig. 4, Chap. IX.

**(38)** In this design M.F.L. is assumed be 3 feet above F.S.L. Thus the whole embankment has to be made 3 feet higher than would be necessary did the full supply level coincide with the maximum. If collapsible crest shutters 3 feet deep were introduced as previously suggested, great saving would result in earth work. The masonry crest of the weir would then be 3 feet lower. The only difficulty in connection with the adoption of such gates is the necessity of an establishment to raise them at the proper time when the reservoir has been sufficiently relieved. Their provision, however, will not effect reduction in the length of the waste weir itself, as would be the case where the deep overfall sluice or low level sluice systems already described were adopted. For the purpose of lifting these gates, light short piers would have to be constructed at, say, 15 feet intervals, with one gate between, and a roadway formed by rolled beams and planks. The gates could be pulled up by attached chains passing over pulleys to a vertical drum or windlass revolved by gear or handspikes, which could be a fixture in each span; or a travelling winch running on rails could be adopted. A description of the crest shutters, 4 feet deep, of the Chenab Weir is given in Min. Pro. Inst. C.E., Vol. CXIII. These are hinged at base and raised from a platform above the weir. See also Fig. 35 (post).

The weir piers could project over the face or rear of the weir walls, so as to afford a sufficiently wide roadway without unduly widening the top width of the weir wall. This has already been exemplified in Fig. 7, Chap. X.

**(39)** Where local circumstances admit of it, the cutting of a wide by-wash will answer the purpose of disposal of surplus water from a reservoir. A very good example of such a work is illustrated in Fig. 15. The by-wash is a cutting 800 feet wide taken through the crest of hill which forms one boundary of the tank embankment; assuming no drop at the head, the slope is, roughly, about one in 700, or 1·4 per 1,000. With a 3 feet depth of water passing through, $A = 2{,}400$ square feet, $WP = 806$, $R = 3$ nearly, and $100\ RS = 6{\cdot}6$ nearly, whence $Q = 2{,}400 \times 6{\cdot}6 \times {\cdot}75 = 12{,}000$ cubic feet per second. ·75 is the coefficient $c$ obtained from the Tables for a bed slope of one per 1,000, which will answer for all greater slopes (*vide* "Hydraulic Manual"). If a depth of 5 feet were adopted, the discharge through the by-wash channel would be $Q = 4{,}000 \times 8{\cdot}5 \times {\cdot}78 = 26{,}500$ cubic feet. What discharge the channel is intended to carry is not known. According to Table II., Chap. V., the discharge per foot run over a weir with free overfall is 17·3 cubic feet when $d = 3$, and 37·3 cubic feet when $d = 5$. The lengths of waste weir required to discharge the same amounts previously arrived at, viz., 12,000 cubic feet and 26,500 cubic feet, would be $\frac{12,000}{17·3} = 694$ in the first case, and $\frac{26,500}{37·3} = 710$ in the second case, the depths of water being the same in either case. Thus we see that with this steep slope the discharge is about seven-eighths of that with a free overfall.

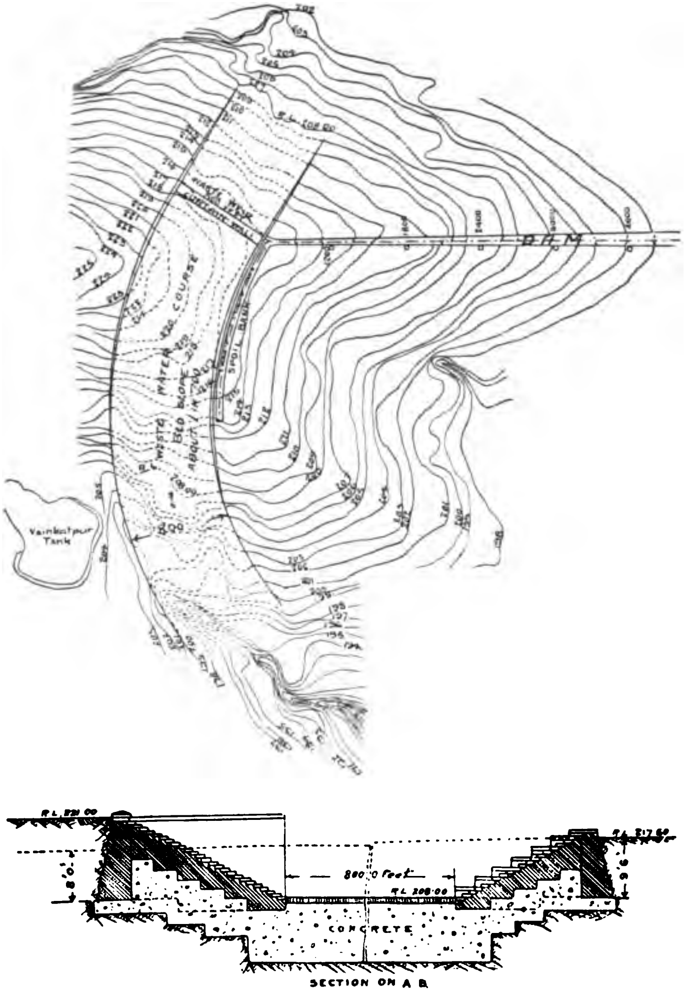


FIG. 15.

A longitudinal section of the weir wall is given below; a transverse section through the weir floor and curtain would be much more useful, as for want of this we are unable to state whether the floor is at the same level as the crest of the weir, which is then really not a weir but simply a protective curtain wall, or whether there is a drop. The probability is that there is a drop of 5 feet. The discharges would then be one-eighth greater than previously calculated.

If the ground is rocky, but will eventually wear away with the current, a good plan is to make but a shallow excavation, allowing the torrent to cut its own channel. If this is the case a weir should be built complete at the head of the work, but sunk in the ground, there to await the retrogression of levels in the escape channel, which may not be fully developed for some years.

Distinctly the best type for by-wash falls in cutting is that illustrated in Fig. 24, Chap. I.

**(40)** In some cases, where the ground is favourable, rapids are a cheaper construction than a series of masonry drops, and are just as efficient. A rapid can often pass as much water over its crest as if the overfall were free.

Rapids are constructed either on a continuous slope, the inclination of which varies with the contour of the ground, or else are built in steps. Water cushions are often formed in each step by building a narrow wall above the tread at its edge; this holds up water over each step, and serves to dull the velocity of the current. Some examples are given in " Irrigation in India " and in " Waterworks Engineering."

**(41)** In the United States many excellent examples of storage reservoirs are to be met with, formed by damming up rivers, mostly in the upper reaches, where rock exists in the flanks as well as in the bed of the river. Many such sites are not suitable for canals to take off. This is effected lower down in the course of the river, where another weir or dam of smaller dimensions, acting only as a diversion weir, is built with the concomitant canal Head and possibly scouring sluiceway. In such cases the reservoir is tapped, as was effected in the Periyar project, by a tunnel, the entrance to which is controlled by sluice gates, or by a deep diversion cut in the solid rock at one flank of the weir, as in the case of the Minidoka Canal Head Works. Where canals can take off they often have to run in a winding course, cut as a bench in the hill side, starting parallel to the course of the river instead of at right angles to it. This arrangement is rendered necessary by the configuration of the ground.

In Fig. 16 we have a section of the dam with a site plan of the head works of the Minidoka Canal, the Head Regulator of the canal of which has already been illustrated and commented on in Chap. VIII. The dam in this case is a combined loose rock and earth dam 60 feet high, fully explained by the section (Fig. 16a). The diversion channel, whereby the reservoir is tapped, is shown cut in the rock on the left flank of the weir. This is closed by five gates, 8 feet wide and 10 feet high, each operated by a pair of geared screws. The sill of these sluices is at R.L. 102·00, or 52 feet below crest of dam.

As is frequently the practice, the waste weir is situated on top of one or both the shoulders of the hill adjoining the canyon, a spillway being excavated if necessary to a depth sufficient to bring the crest of the waste weir 5 feet or more above the highest point of the spillway. If the waste weir is not raised well above the spillway, its free discharge will be greatly hindered by the shallowness of the approach channel. In the present instance no excavation seems to be requisite, as from the section given, the natural surface appears to be 14 feet below the weir crest and 18 feet below that of the dam. On this the weir is built to a curve on plan, following the highest ground line.

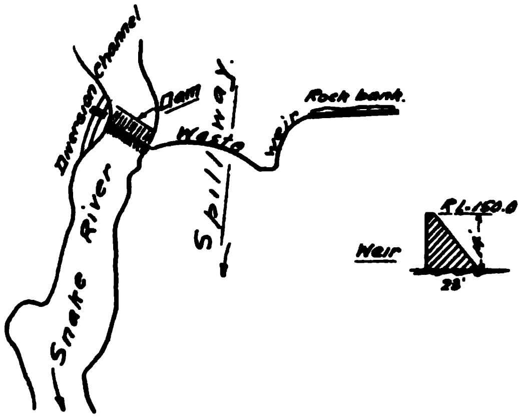


FIG. 16.—Minidoka Site Plan.

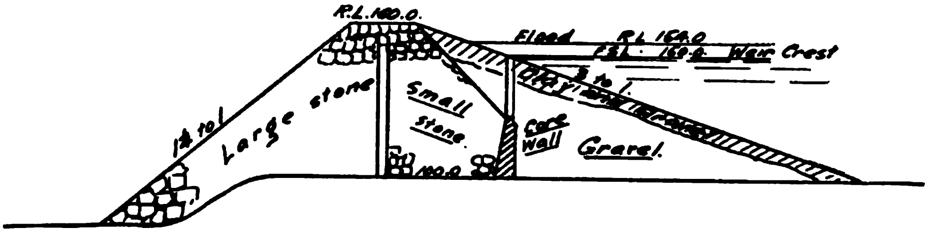


FIG. 16a.—Minidoka Dam.

(42) A somewhat similar case is illustrated in Fig. 17, which is the site plan of the Roosevelt Dam across the Salt River in Arizona. This work is under construction at the present moment.

The dam is curved on plan, but is considered as a gravity dam, not depending on its curvature for stability. A roadway 13 feet wide inside parapets is provided along the crest and continues across the spillways on either flank over two large span bridges 200 feet in length.

These spillways are excavated in the solid rock, and the stone thus obtained will probably be used in the dam. They both start at about R.L. 190 and slope up to the weir, where the floor is at R.L. 205, and then immediately begin to fall down to R.L. 155, where the rapids merge in the walls of the canyon. The waste water is thus safely conveyed away, clear of the dam. These waste weirs are each 200 feet long and 5 feet high. The reservoir will be tapped by a tunnel in the solid rock 400 feet in length, which will be controlled by a sluice tower head with six gates. This tunnel will act as a silt scouring as well as a supply sluice for the canal works lower down the Salt River.

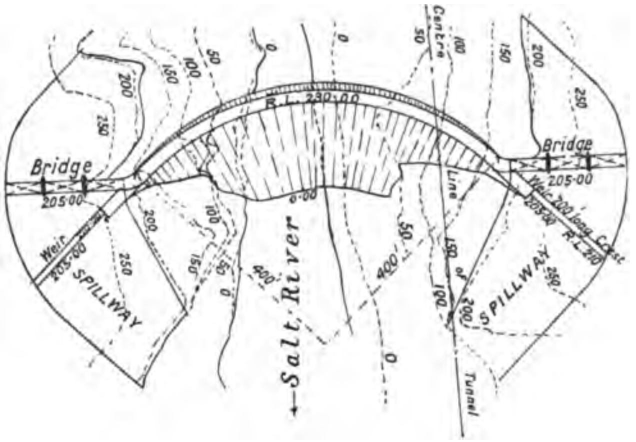


FIG. 17.—Roosevelt Dam, Site Plan.

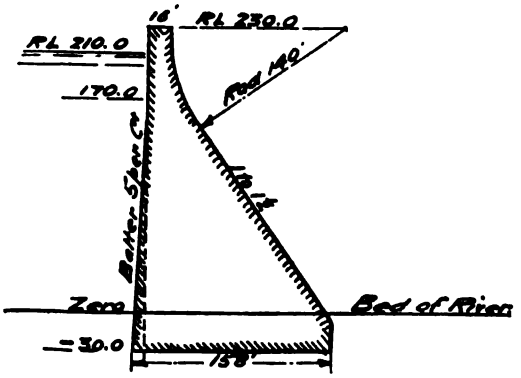


FIG. 17a.—Roosevelt Dam.

(**43**) The section of the Roosevelt Dam is given in Fig. 17a; its outside height is 260 feet. It will therefore rank among the highest dams in the world. A remarkable feature of this section is the fact that it does not depart sensibly from the elementary triangular profile mentioned in Chap. II. Consequently it is calculated that the maximum permissible stress on the masonry is over 20 tons, *i.e.*, double the usual. Taking $H$ the depth of water above base as 250 feet: from the formula $H = \frac{\lambda}{w(\rho + 1)}$ we have $\lambda = H \times w(\rho + 1)$ or $\frac{250 \times 13}{36 \times 4} = 22$ tons, $\rho$ being $2\frac{1}{2}$ and $w$, $\frac{1}{36}$ ton.

The base width, according to the formula applicable to " Low " dams, is $\frac{H}{\sqrt{\rho}} = 250 \times \frac{2}{3} = 166$ feet. It is actually 171 feet, a close correspondence. In this base width, the portion very improperly cut off the extreme toe below zero, is included. A case like this is one suitable for the employment of steel reinforced cement concrete. The Croton Dam is on same principle.

(**44**) A further example of a reservoir work is the Folsom Canal Weir built across the American River in California (*vide* Fig. 18). The remarkable disposition of the head works is due to the necessities imposed by the site. The river is crossed at right angles by a masonry weir, which for a length of 180 feet has the crest lowered. This gap can be closed by one long hinged truss gate, operated by hydraulic presses, details of which are not available. The higher portion ends on the west side with a Head Regulator, the canal of which takes off at right angles to the axis of the weir. On the east side the weir is turned in a curve of 90° and thus continues parallel to the river up to another canal Head, that of the East Canal.

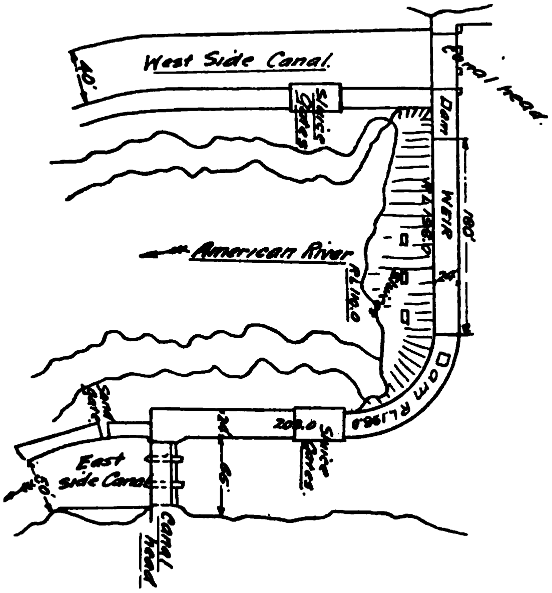


Fig. 18.—Folsom Canal Head Works.

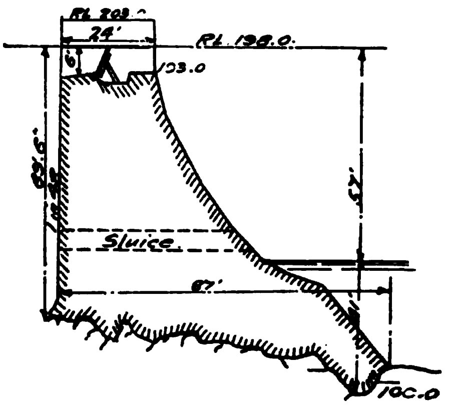


Fig. 18a.—Folsom Weir.

The crest of the higher part appears to be only 5 feet above that of the weir crest, *i.e.*, on the same level as the crest of the weir shutters, with the evident intention that in case of high flood the long arm of the weir could be overflowed and thus utilised as an emergency outlet for flood water. The minimum flood level is 30 feet above weir crest or at R.L. 225.

In both canals sluice gates are provided for the purpose of removing silt, while there are three culvert sluices in the body of the weir itself at a low level in order to scour deposit occurring in rear of the weir wall.

The Eastern Canal Head is a very massive structure, and has been already illustrated and analysed in Chap. VIII. This Head is provided with what are termed sand traps, that is, silt is deposited in the sand traps and carried away by the sand gate shown at the side. This arrangement is all the more necessary as the scouring sluices are situated too far away to much influence deposit in front of the canal head work, and by the lifting system adopted, the water can only be admitted at the bottom of the gates; thus any accumulation must be washed inside the canal.

**(45)** The section of the weir wall is given in Fig. 18a.

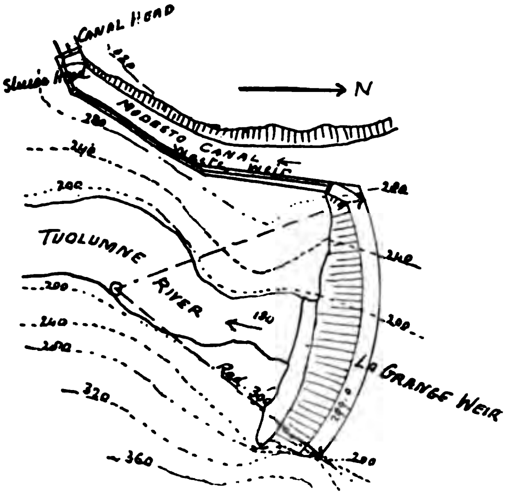


FIG. 19.—Site Plan, La Grange Weir.

The flood level being 30 feet above crest the latter must be submerged or nearly so. Assuming the level of tail water to be at crest level, *i.e.*, R.L. 193, $d$ will $= 30$ feet, and by formula (4) (Chap. III.) the base width at level of the heel of the weir wall should be $\frac{H + \cdot 6d}{\sqrt{\rho}}$ or $\frac{65 + 18}{\sqrt{\rho}} = 83 \times \frac{2}{3} = 55$ feet. This is just about what it would scale if the excrescence at the toe were neglected.

The top width should be $\sqrt{H_1} + 1 = \sqrt{95} + 1 = 11$ feet nearly. On account of the shutter an increased width of 15 feet might be allowable, but 24 feet as adopted is clearly extravagant, particularly as it is carried round the bend of the *L* where a crest width of 5 feet would be ample, the depth here being quite inconsiderable.

The R.L. of the canal bed on either side will be about 191·00, and that of the east side approach channel 3 feet lower, or 188·00. Top of the Head Regulators will be at R.L. 228·0 These reduced levels are based on the assumption that the flood level is 30 feet above the lowered crest of the weir, or at R.L. 223·0. This work is really not a storage, but a high diversion weir.

**(46)** Fig. 19 is the site plan of the La Grange Weir in California and connected head works. A section of this weir has already been given in Fig. 16, Chap. III. The arrangement of the canal off-take is similar to that of the east side canal of the Folsom Dam (Fig. 18). The canal runs parallel to the river on a bench cut in the shoulder of the hill, with a flank wall on the left side towards the river. This flank wall acts as a waste weir and is practically a continuation of the main weir, but at right angles to it. This subsidiary waste weir terminates with a scouring sluice head, close to which, at right angles, is the Modesto Canal Head Regulator. This disposition is clearly a better one than that at the Folsom Canal, where the sluice head is placed not close to the canal head, but higher up the approach channel near the weir. The clearance of all deposit in the approach channel, which also acts as a sluice way, is thus assured, which is not the case with the Folsom Canal Head. On the left weir a tunnel is bored through the rock for the supply of the Turlock Canal (not shown on site plan). The Croton Reservoir Dam and the Dhukwa Weir are treated at the end of this Chapter.

### *Reservoir and Tank Irrigation Outlets.*

**(47)** Next to pipes laid through an embankment, which as a rule are only suitable for small tanks upholding 6 feet to 8 feet of water, the simplest form of outlet is that illustrated in Fig. 20. This consists merely of a narrow bridge opening, which is regulated either by double wooden gates either superimposed or moving in double grooves, or by sleepers or baulks of wood, which are slipped into the grooves and removed or replaced one by one as required. The advantage of the latter system is its extreme simplicity and ease of manipulation. The water being drawn from the top and having a vertical overfall, the velocity of entry is mostly absorbed. On the other hand, when draw gates are used, the water issues with considerable velocity, the opening acting as a sluice under a head of water. The use of double or triple gates obviates this to a considerable extent, as when the top gate is completely withdrawn the conditions become those of an overfall.

In Fig. 20 the opening is 5 feet wide, the floor for the length of the abutments is formed by an invert, a suitable arrangement in this case, as the abutments can be considered as a masonry beam supported at both ends and subjected to a load increasing from the top with the ordinates of the triangle of earth pressure (*vide* Chap. I.), and so can be designed of light section.

The arch is placed so as to be clear of the water spill at full supply. The wings are of the ordinary sloping crested type, and their bases are divergent, one in twenty (or the vertical batter multiplied by the slope of crest) from a parallel to the axis of the work. On both sides the wings have a crest slope of two to one, that of the embankment on the rear side being reduced gradually from its normal inclination of three to one to the steeper slope. This intermediate portion of bank should be pitched. The rear wings slope right down to the ground level, which is the most economical arrangement, while the fore wings terminate in short returns so as to form a junction with the bank of the escape channel or canal.

**(48)** When the water is deeper than about 10 feet, this type of outlet will not answer. It will be more economical to adopt a barrel culvert for the outlet, thus abolishing the abutments. The wings, however, will remain, with the addition of a breast wall on the fore side as connection.

Sluice outlets of this description are illustrated in Figs. 21 and 22, in both of which the arrangements are similar.

The dimensions of the culvert are determined by the following considerations:—

Firstly: They should be sufficiently large to admit of a man entering them for examination or repairs.

Secondly: The maximum mean velocity of the current through the culvert should not exceed 5 feet per second. In Madras 15 feet per second is allowable.

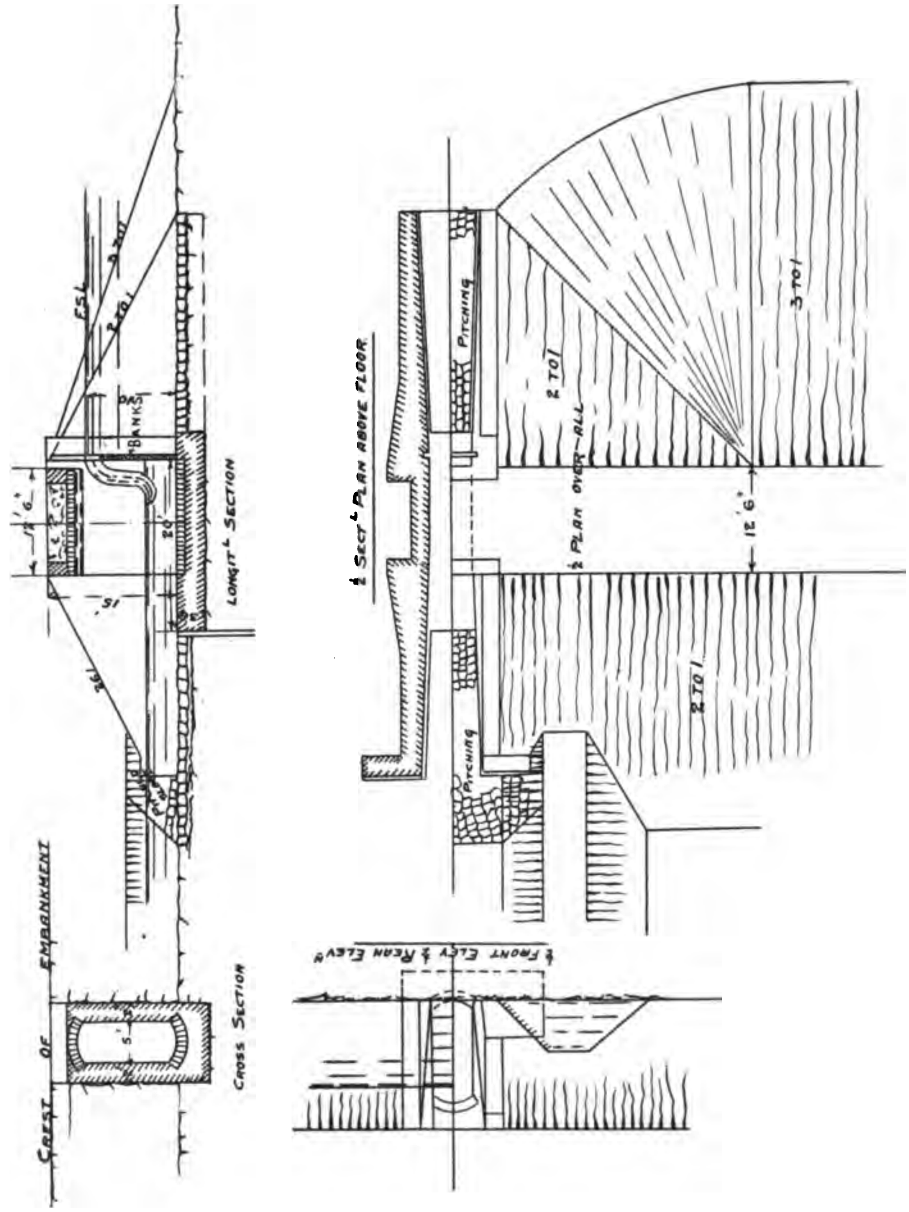


Fig. 20.

In Fig. 21 the sluice gate is 2 feet square; the head of water, $H$, is 7 feet, *i.e.*, considering the opening to be just submerged, consequently the velocity of issue, according to Formula (1), Chap. V., will be $c\sqrt{2gH}$, or taking $c = \cdot 62$, the velocity of issue will be $\cdot 62 \times 8\cdot 025 \times 2\cdot 646 = 13$ feet per second nearly, and the discharge $Q = AV = 4 \times 13 = 52$ cubic feet per second. To pass this discharge at the limit velocity of 5 feet per second the sectional area of the culvert barrel will have to be $\frac{Q}{V} = \frac{52}{5} = 10\frac{1}{2}$ square feet nearly. The culvert then should be $3\frac{1}{2}$ feet wide × 3 feet high, to half-way up the arch.

In the design in question, however, the maximum discharge was limited to 30 cubic feet per second, so that the sluice gate will never be fully opened when the water level is at full supply or tank level, and the area of the outlet culvert was designed accordingly.

It is, however, too small for inspection purposes, being only 3 feet × 3 feet outside dimensions, and would have been better if made the larger size.

In this design one set of grooves is provided distinct from the gate for the purposes of blocking the outlet by baulks to admit of inspection and repairs to the gate. The shutter or gate should be of cast iron strengthened with gussets on the outside, and the grooves should likewise be of cast iron. Grooves made up of angle or

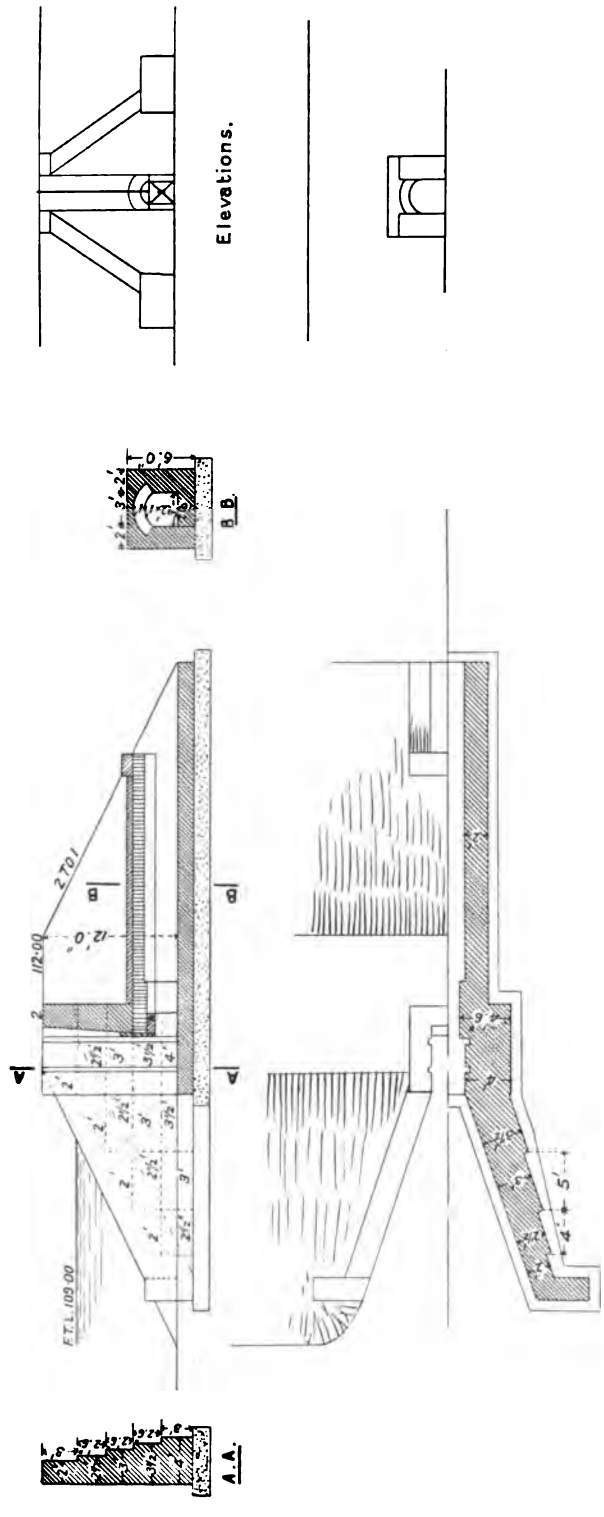
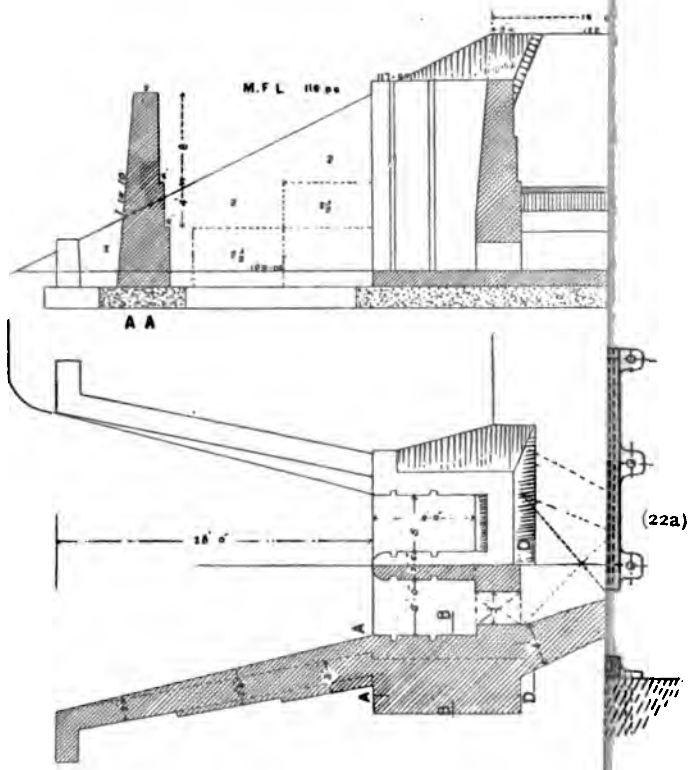


Fig. 21.



(22a)

channel iron, though sometimes used, are by no means satisfactory. The lifting apparatus should be screw gear, the best form of which is illustrated and described in Chap. XIV. The head of the sluice should consist of a square cast iron flanged box built into the masonry, the hollow spandrel between it and the relieving arch being built up with brickwork in cement.

**(49)** Fig. 22 is a representation of a similar work, but on a larger scale. The sluice openings are double, 3 feet × 2½ feet in size. In order to save masonry, the projecting piers and abutments terminate at *R*117, or 1 foot above maximum tank level, and space for the working platform, which is usually of wood, is obtained by sloping back the embankment. Access to this will be afforded by a step-ladder. Two sets of grooves are provided in order to enable either of the gates to be examined and repaired with a head of water in the tank. This can be effected by filling and consolidating earth in the space between the sets of baulks placed in the grooves. Figs. 22a and 22b are sketches of the gate and its frame. The sluice head openings can be lined with cast iron plates or else built of ashlar. The roof can be made of rails or rolled beams set in cement concrete, or ashlar slabs could be used. Each of these sluice openings, with a maximum free head of 13 feet, will discharge $Ac\sqrt{2gH}$, or $3 \times 2\frac{1}{2} \times \cdot 65 \times 8\cdot 025 \times 3\cdot 6 = 140$ cubic feet nearly, or with both open, 280 cubic feet per second. The area of the culvert when full is about 28 square feet, so that the velocity of passage will be 10 feet per second with both sluices open. The channel to carry this discharge would have to be of 20 feet base width at least.

**(50)** In order to neutralise the velocity of issue a common practice is to build a square chamber in the rear of the culvert with bed sunk well below the level of the floor, and a low weir in front raised above the floor. The increased waterway thus given absorbs the velocity and the current issues from the chamber at a moderate pace. The outline of this is shown dotted on the plan and elevation in Fig. 22. The banks and bed of the issuing canal would have to be pitched for a considerable distance below the termination of the floor and wings of the sluice, unless some such remedy were adopted. The design was, however, made under the assumption that only one sluice would be opened when supply was at the maximum, the greatest allowable discharge being about 140 cubic feet per second. This would give a velocity of 5 feet through the culvert. Under such circumstances the well would be unnecessary.

**(51)** The disadvantages of draw gates to reservoir sluices is that careful regulation is required to prevent waste of water, and the screw lifting apparatus is expensive. With deep water the gates require great pressure to force them down. The coefficient of friction in rusty gates certainly equals unity. This last objection can be obviated by introducing anti-friction rollers in the gates. The groove of the frame in this case will have to be made wider and deeper, or else could be abolished altogether. In any case, the use of vertical stanching rods will prevent leakage at the sides, and a horizontal one at the top between the upper edge of the gate and the sill. By this simple arrangement the frictional resistance would be reduced by 70 or 80 per cent., thus enormously relieving the pressure on the screw gearing, which could be made of a very light section in consequence. An example is given in Fig. 22b.

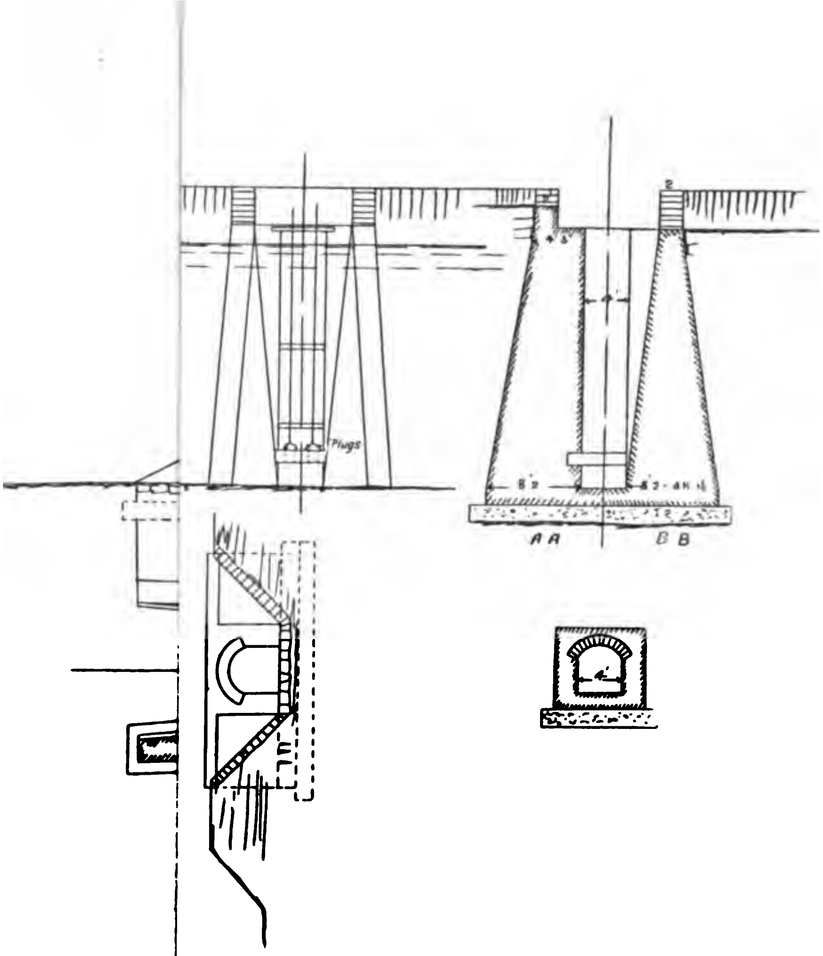
(**52**) In the Madras Presidency a system has been successfully employed for many years, which abolishes all frictional resistance, the pressure of water alone having to be met, and that on a very small area, and further enables very accurate regulation of the water to be effected. This system consists of admitting water into the culvert by means of circular holes cut in the flat roof of a chamber built in front of the sluice head, which holes are closed by conical plugs of wood.

Fig. 23 contains a design illustrating this construction. Before proceeding to discuss this design we will give the following extract from the "Madras Irrigation Manual" relative to this subject.

"Taking into account facility, simplicity, accuracy and safety, there are no better means of regulating the discharge of irrigation water from tanks of the ordinary kind than plug holes provided with suitable plugs. To secure convenient and fairly accurate regulation it is necessary that (1) the holes should be of a suitable size and sufficient in number; (2) proper plugs should be provided and a well-cut seat made for the shoulder of the plug; (3) the plugs should be regulated from a platform always accessible; (4) it should be known what the discharge is at any given time, and with the tank water at any given level; and that the way to secure any required discharge should be readily ascertainable. In order to facilitate calculation and comparisons, it is convenient to adopt one ratio of coning for all plugs, and this has been fixed at one in four; also to select the following diameters of plug holes for general use, viz., 4, 6, 8, 10 and 12 inches.

"It will often be found that a very small expenditure on the improvement of an existing sluice will obviate the necessity for constructing additional sluices, which may be asked for by the cultivators. In addition to the plug holes, there is usually and properly a low level vent, which is closed and regulated by planks or a shutter sliding in grooves. The regulation of this lower vent cannot ordinarily be conveniently managed when the water is more than 2 feet deep on the plug stone, corresponding to about 3½ feet on the sill of the sluice. It is therefore desirable that the whole area of cultivation under a sluice should be capable of being efficiently supplied through the plug holes alone when the surface of the water is 2 feet above the plug stone.

"Again, although 2 cubic yards per acre per hour, or 1 cubic foot a second for 66 acres, is usually adequate as an average supply, this quantity, when much of the land is being prepared for cultivation, will be by no means enough. It is desirable, therefore, that the plug holes should be capable of discharging much more water at times, and this can be provided for by allowing one or two large, or several smaller, plug holes so as to admit of the discharge being varied according to circumstances.



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"It is desirable to provide means for raising the plugs to a height just sufficient to secure a full discharge, and no more, and this will be the case when the bottom of the plug is raised to a distance, equal to the diameter of the hole, above the plug stone. An iron bar (round), passed through the plug and secured thereto by a key or cotter at top and bottom of the plug, is the best arrangement for the lower part of the attachment, whether a rigid bar be used throughout or a chain be the connection at the upper part with the platform. A cross-bar with a hole or a collar is the most suitable means of limiting the lift of the plug; and a second cross-bar should be placed just below the top of the rigid part of the lifting apparatus when a chain is used for the upper part. The plug will then be always vertically over the hole to whatever height it may be raised. When a rigid bar is used from the plug to the platform, the upper part should be flat and should be pierced with holes at short intervals (3 inches will usually be appropriate), as generally with some small and some larger holes in the plug stone the regulation can then be effected with sufficient accuracy. Similarly, when a chain is used for the upper part of the lifting gear, the links may be 3 inches long at the part within the range of the lift.

"When the plugs are of large diameter and the depth of the water great, they cannot be lifted by one or two men, and the provision of some sort of mechanical lifting gear will be necessary. This may be either a screw or a windlass or an overhead bar with small pulley tackle, either of which would enable the regulation to be readily and easily effected. A windlass should have a ratchet with pawl or detent, to keep the former in any required position.

"The plugs should be made of hard dense wood, and should be turned in lathe, so as to be true to gauge. Heart of Tamarind or of Vagay (Tamil), Darasippa (Teluga) makes good plugs. They should have iron flush straps at top and bottom of the coning to prevent splitting. The shoulders of plugs should be lined with leather, the thickness of the layer varying from ¼ inch to 1 inch, according to the size of the plug and the depth of the water (maximum). This layer of compressible material will diminish leakage and reduce the risk of fracture of the plug stone, should the plug be accidentally allowed to drop suddenly."

Drawings of different sized plugs are given in Fig. 24.

(**53**) The remarks above quoted concerning lifting apparatus require, it is considered, some modification. All plugs, small and great, should be lifted by screw gear. The rods for the larger sized openings under considerable pressure should be hollow, of gas or water piping. At the end of these rods, which should terminate above M.W.L., a brass female screw should be fitted. This is engaged by threads cut on a solid round bar; near the upper extremity of this latter a circular thrust plate is forged which works in a thrust box fitted with brass bearings, above which is the square rod head which takes the handle. Thus the power is applied to the male screw, which when revolved enters the hollow pipe, drawing it and the attached gate or plug upwards; in the same way by reverse action the pipe rod is forced downwards.

This method of screw gear is far superior to that commonly employed, which is to adopt a solid rod threaded at the upper end. This screwed end is embraced at the top by a short female screw with a thrust flange and box. In this case power is applied to the female screw and the solid rod passes through it, rising above. This system necessitates a long, heavy, expensive solid rod, and, further, the screwed end rising above the platform is exposed to the weather; whereas in that first described, the screwed solid rod is only of a very short length, viz., that of the vertical play of the gate or plug, and can easily be replaced when worn, and the transmitting rod, being hollow, is well suited to resist tortional as well as compressive strain. The solid screwed rod is further protected from the weather and from dust, etc., as no portion appears above the platform. Both these systems are illustrated in Chap. XIV., Figs. 3 and 4. The second system is only suitable for the smaller sized plug holes and should never be adopted for sluice draw gates.

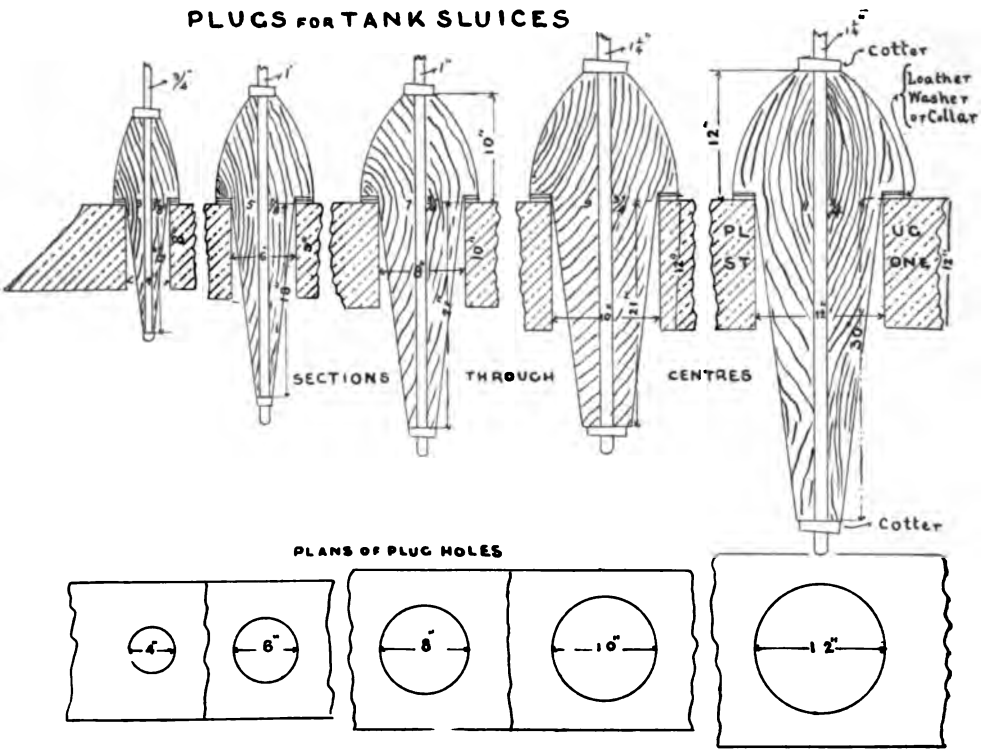


FIG. 24.

(**54**) Another point requires notice. The fore bay chamber is made only 2 feet in height and is shown such in the drawing in Fig. 23. This limit in depth is due to the want of proper grooves and lifting gear; the grooves arrangement shown in the original plan in the "Madras Irrigation Manual," which has been only followed as far as the section of the culvert and fore chamber, is exceedingly primitive. With proper iron grooves the wooden gate, even without rollers, could easily be lifted by means of an ordinary wooden windlass, under a very much greater mean head than 3 feet, which is apparently the limit. It is considered that the chamber should be at least 3 feet high to enable it to be inspected, except in the case of very small tanks.

(**55**) The arrangements in design of the sluice in Fig. 23 show some differences from that in Fig. 22. The fore wing walls, as also the breast wall, are run right up to the crest of the embankment instead of stopping short at the platform level, which is undoubtedly a better arrangement. The amount of masonry saved in the former design is not much. The abutments shown in section *AA* have to be thicker at the top than the normal 2 feet, but are proportionately reduced at the base so as to keep the average width at a quarter the height, which is about the same ratio as in the trapezoidal profiles with vertical faces indicated in Table I., Chap. I., and exemplified in the section *BB*. The breast walls in both cases have a base width of about $\cdot 25H$, which is quite sufficient considering the extreme shortness of the unsupported length. The section of the wings in Fig. 23 have a fore and aft batter in accordance with the model profiles exhibited in Chap. I., whereas in the earlier design (Fig. 22) they have vertical stepped backs, an arrangement not so scientifically correct as when the base thickness at every point bears the same definite ratio to the height.

(**56**) As already noticed, type sections of plugs of different diameters, taken from the "Madras Manual," are given in Fig. 24. The details of the rods and screw gearing are omitted, as the arrangement is very simple. Cross-beams of wood are shown in Fig. 23 on the sides of each set of rods, on which a bracket bearing is bolted, which, besides acting as guide, affords the necessary support against flexure when under compression. To aid torsional resistance the rods or pipes should be furnished, at or near the hollow screw, with arms tightly clamped or screwed on, the ends of which arms run in vertical guides.

(**57**) To the critical observer it will be apparent that the amount of masonry involved in a high sluice head, as exemplified in Fig. 23, is very considerable. The only usual alternative is to run the culvert right on to near the fore toe of the embankment, and there connect it with a tower. This construction is, however, seldom adopted, except for much larger depths of water, as there would be no saving in cost. The author considers that for moderate depths, say up to 25 feet, a modification of the watertight tower fitted with sluice openings in the sides, as illustrated later, would be much more economical than the ordinarily adopted design of Figs. 22 and 23, and at the same time be equally efficient. A sketch of this is given in Fig. 25.

(**58**) The principle of this design is that the culvert is produced outwards to near the toe of the embankment. Outside is the closed chamber with draw gate and plugs, just as in Fig. 23. There being no earth to be upheld or water to be kept out, all that is required is an open staging erected over the chamber, from the top of which the draw gate and plugs can be worked, or, in the case of a sluice head, as in Figs. 22 and 23, for manipulating the draw gate. The depth of water is not such that a watertight tower is required at all, the object of which is to limit the pressure on the sluice gate or pipe valve, as the case may be, so that the structure, if of masonry, can be made very light, or simple open iron or wooden staging would answer equally well. The design for this masonry head, as shown in Fig. 25, consists of a U-shaped square well of 3 sides only, with openings in the walls, the back being towards the embankment. This well is covered in at top just above

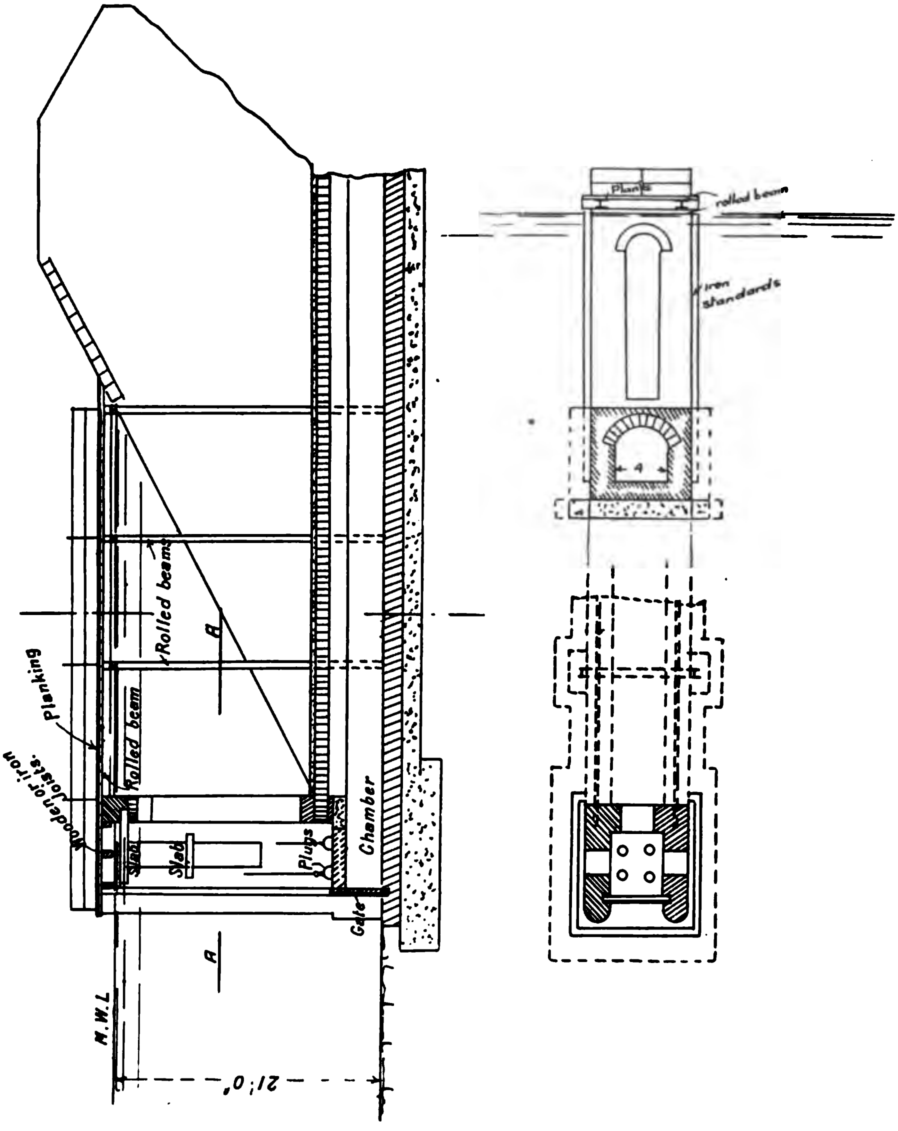
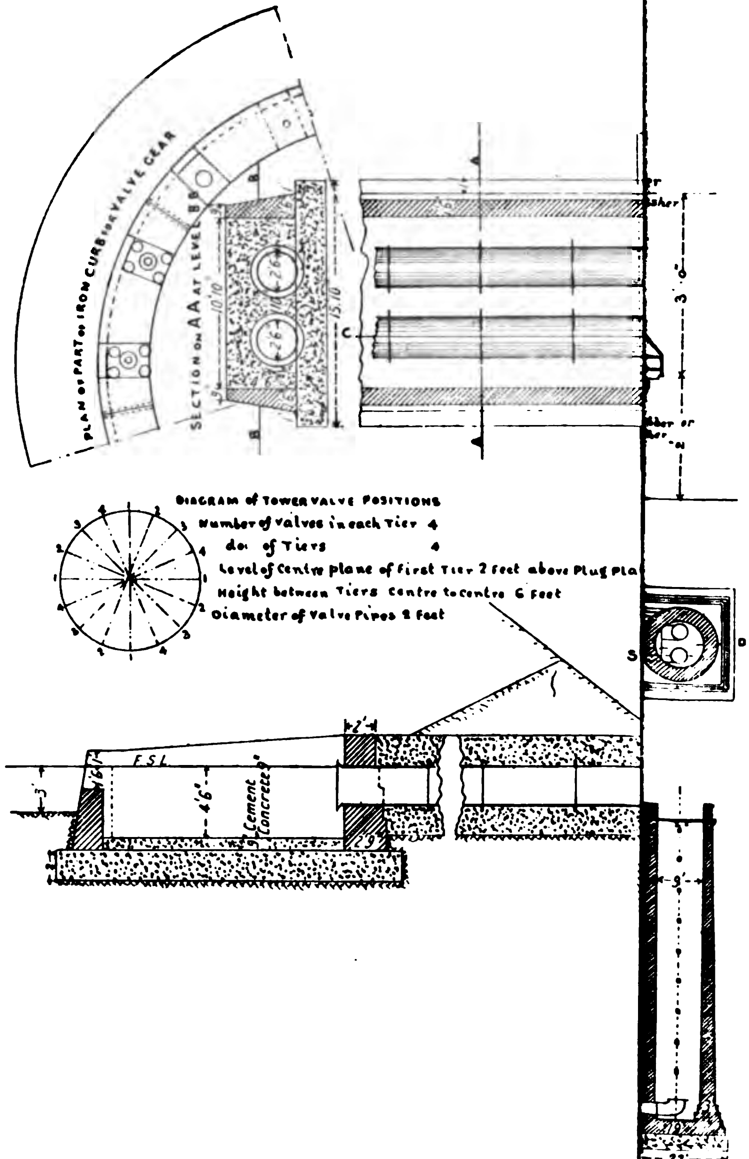


FIG. 25.


PLAN OF PART OF IRON CURB FOR VALVE GEAR
SECTION ON AA AT LEVEL BB
DIAGRAM of TOWER VALVE POSITIONS
Number of Valves in each Tier 4
do. of Tiers 4
Level of Centre plane of first Tier 2 feet above Plug Pla
Height between Tiers Centre to centre 6 feet
Diameter of Valve Pipes 2 feet
F.S.L.
4'6"
Cement Concrete
3'
2'
2'9"
10.10
15.10
3'0"
9'
22'
A
B
C
S
D

p. 359.

M.W.L. by a wooden planked platform, on which the screw gear heads and windlass will be placed to work the plugs and draw gate. The platform, which is isolated from the bank crest, is connected with it by a foot-bridge, supported on columns at intervals, which have separate foundations on either side abutting on the side walls of the culvert; the superstructure being of rolled joists and longitudinals covered by planking.

If necessary, the columns can be counterbraced transversely and longitudinally; the transverse bracing must, however, for obvious reasons, be above the fore slope of the embankment.

Excluding the concrete, the comparative quantity of masonry in the two designs of sluice heads of Figs. 23 and 25, including in the latter the lengthened portion of the culvert, are, roughly, as follows:—

| | | |
|---|---|---|
| Fig. 23 | . | 8,000 cubic feet. |
| ,, 25 | . | 4,000 ,, ,, |

Showing a saving of no less than 4,000 cubic feet effected by the alteration of the design. This comparative statement excludes all parts common to both.

From the value of the masonry saved must be deducted the cost of the planked foot-bridge, which, if built up of old rails, would be but trifling.

**(59)** Fig. 26 is taken from the "Madras Irrigation Manual," and is a design for an irrigation outlet, provided with a tower. The outlet itself consists of two rows of iron pipes set in cement, and the sluice is a plug valve. This design commends itself for its simplicity; the only point to which most engineers would object would be the reduction of the normal fore slope of the embankment from three to one to a curved pitch slope of one to one. It might be reduced to two to one, but certainly not less. The valves in this tower are closed by draw gates. Flap valve openings would probably be equally efficient and less expensive.

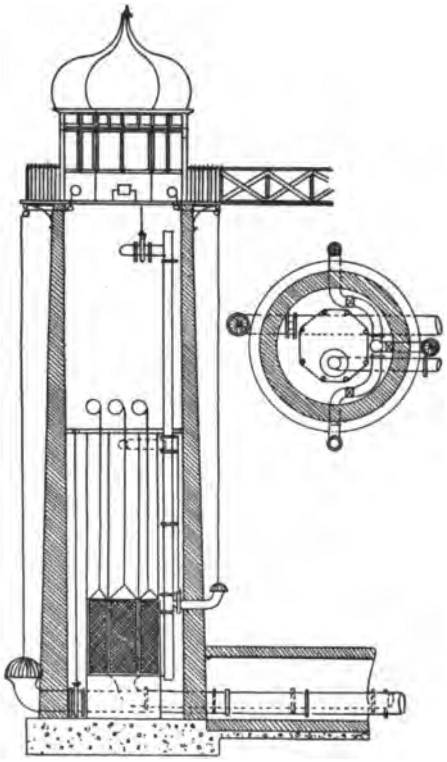


FIG. 27.

**(60)** The ordinary pattern of English waterworks towers is shown in Fig. 27, which is taken from "Waterworks Engineering." The position of the inlet valves to the tower is adapted to draw the supply from any one of three levels in the reservoir, as well as to scour or discharge the water from the bottom. The screen arrangement to ward off impurities would naturally

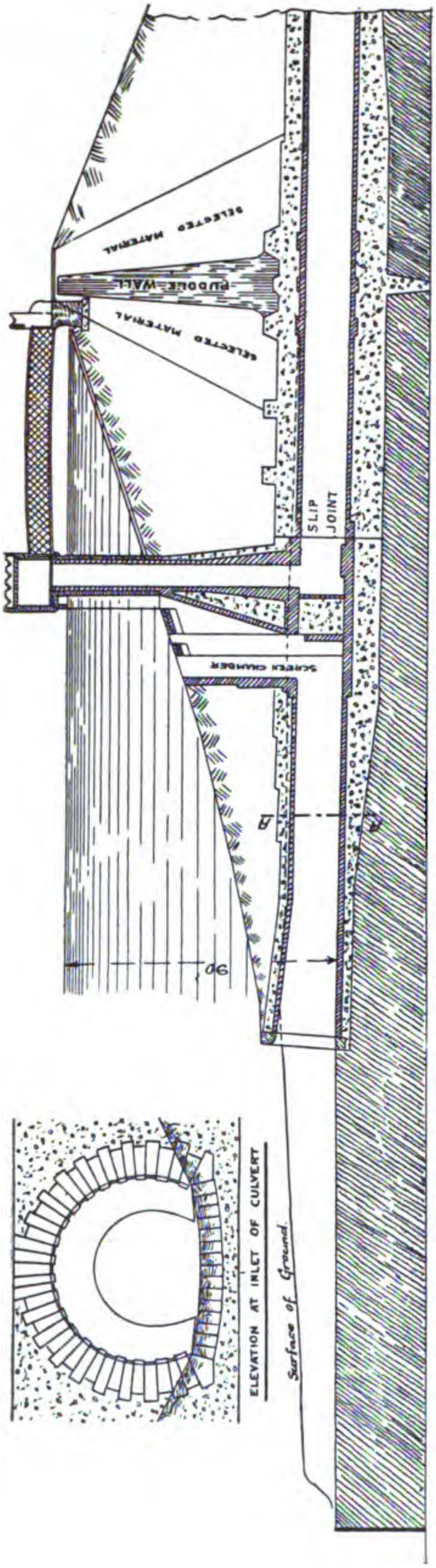


FIG. 28.

not be required in an irrigation work. The dimensions of the wall of the tower, which is circular, are:—Top thickness, 2 feet, which increases ½ foot for every 20 feet in depth below maximum surface level of the reservoir.

**(61)** In order to avoid all earth pressure on the tower this latter is generally built well clear of the toe of the embankment. If, for economical reasons, it is deemed advisable to build it in the middle of the embankment, this can be done, if the section of the tower is increased to meet the stress thus liable to be brought upon it. The increase in thickness of the tower should naturally be on the far side from the embankment. This is well exemplified in Fig. 28, a section of the Hury reservoir embankment, for which we are likewise indebted to "Waterworks Engineering." As will be seen, the culvert is cut in the solid ground, a procedure which should be followed wherever possible.

To allow for uneven settlement in the culvert due to the great pressure, a slip joint is provided. This is shown in the section in question. The method of construction of the slip joint is fully explained in the text, to which the reader is referred.

The best section for a culvert to resist pressure is elliptical, with the shorter axis horizontal; the ratio, as recommended in "Waterworks Engineering," is $\frac{hor.\ axis}{vert.\ axis} = \frac{2}{3}$, the lower third of the section truncated and closed with a flat invert.

The thickness of the arched

wall of the culvert is obtained from the formula $t = b\sqrt{\frac{H}{2}}$, in which $t =$ thickness required in inches; $b =$ the smaller diameter of the culvert in feet; $H =$ height of superincumbent earth in feet.

**(62)** The subject of pipe outlets from small tanks will now be dealt with. These usually consist of earthenware pipes similar to the "Kolaba" pipes used in canal distributaries, with either spigot and faucet joints or else collar connections. In localities where earthenware pipes are difficult or expensive to obtain, excellent pipes can be made of fine cement concrete, which is stronger than earthenware, and can be moulded into any required shape. In the Ceylon Irrigation Department conical cement pipes are used. These are manufactured on the spot. While on a visit to Anuradhapura some twelve years ago, the author inspected the manufacture of these pipes, details of which are given in Fig. 29.

As will be seen in the section of bank, the pipes are run up the fore slope of the embankment, and are removed and replaced by hand in accordance with the level of the water in the tank and the head required. This is a far superior arrangement to the ordinary pipe head at bed level, the closure of which is commonly effected by a man diving down under water and stuffing a handful of straw into the mouth of the outlet. Another arrangement for regulation is to erect a light wooden open staging over the outlet head. The conical pipes are then fixed vertically between three poles driven round them, and the cultivator standing on the cross-bars of the staging can remove or replace the pipes one by one as may be required, leaving the spare ones on the staging or on the adjacent bank.

**(63)** When the height of the embankment is not great, corrugated or sheet iron piping could be used with advantage in place of a masonry culvert. The line of piping would have to be provided with large flanges at intervals to stop the creep of water, and should terminate at either end in a proper masonry fore-and-aft bay. It could be set in puddle or concrete. A chamber with plug holes or a flap valve over the pipe head should be added on the face; also, a wooden or iron staging carrying the lifting apparatus.

**(64)** When a tank embankment dams the course of a large stream which in flood time has to be passed through the reservoir, the question of the possible prevention of silt deposit is a serious one. The river risen in flood pours water heavily charged with sediment into the reservoir, which sediment must nearly all deposit within it, thus gradually but surely diminishing its effective capacity. Scouring sluices, however powerful, will but palliate the evil, as their action is but local in its effect. In such cases the best remedy undoubtedly is to construct some works on the river before it reaches the area of the tank, whereby in heavy floods only the required proportion of the full flood discharge will be allowed to enter the tank, the rest being disposed of elsewhere into another drainage line. In cases when the whole discharge

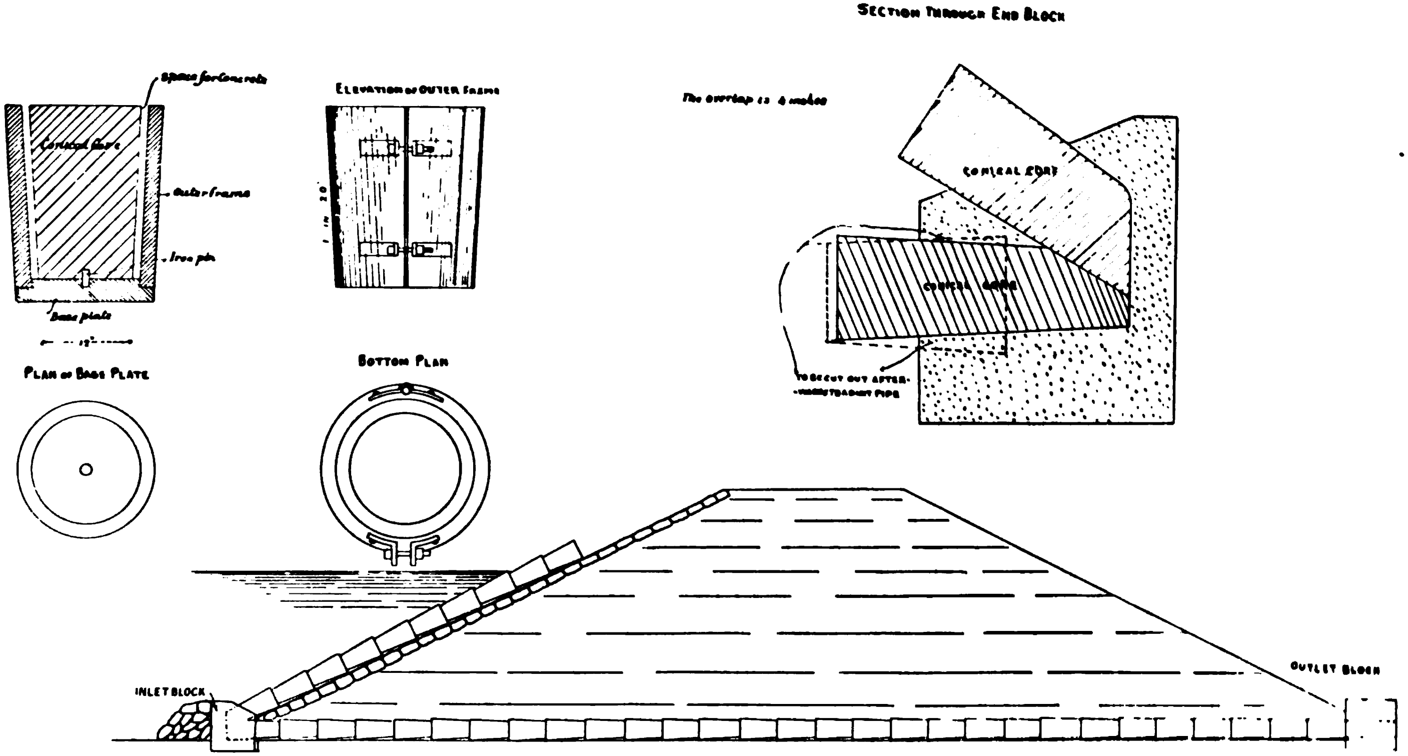


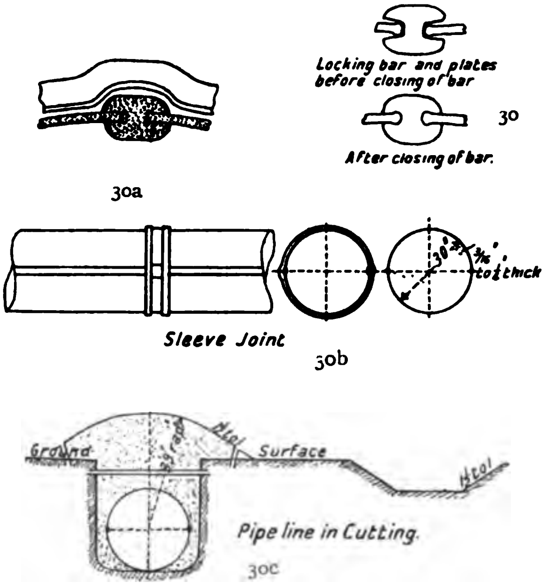
Fig. 29.—Ceylon Concrete Tank Outlet Pipes.

is required to fill the tank, the current can often, by means of a weir, be forced to spread over an area of flat country on either side, and made thus to deposit its silt on the land before it reaches the depression in which the reservoir is situated.

In a large reservoir a floating dredger will successfully keep down silt deposit. The possible prevention of silt deposit is a matter which should be carefully thought out on the initiation of any new reservoir project, as many old works have been rendered quite useless by silting up.

(**65**) Since the above was written, an interesting paper on the Coolgardie Supply Reservoirs has appeared in Vol. CLXII. of the "Minutes of the proceedings of the Institute of Civil Engineers," from which the following information regarding a very recent work, the largest ever constructed of its kind, is deemed of interest :—

To supply the mines at Coolgardie and Kalgoorlie, situated 350 miles from the West Coast, an immense reservoir was formed in the rainy belt near Fremantle, on the West Coast, having a maximum capacity of 4,600 million gallons. The surplus falls over a weir which is the highest ever constructed. Its section is given and discussed in Fig. 30, Chap. III. From this reservoir water has to be pumped, by eight pumping stations, the unprecedented distance of 360 miles.

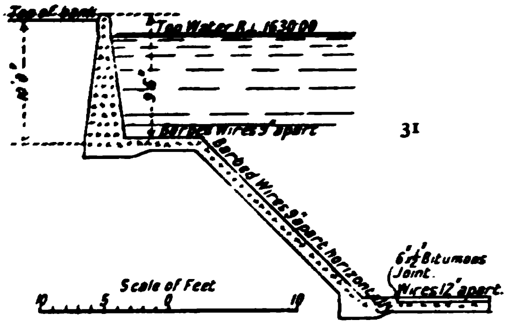


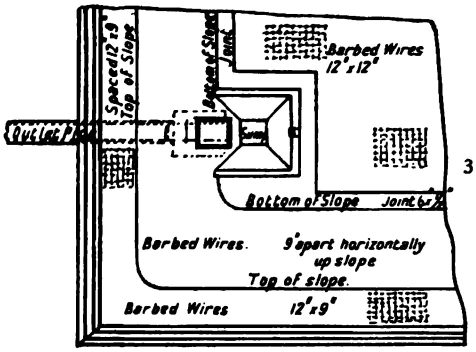
FIGS. 30, 30a, 30b, 30c.—Coolgardie Pipe Line.

(**66**) Cast iron pipes being clearly out of the question, steel pipes 28 feet long and about 2·6 feet diameter were used. These are of the locking-bar pattern. To quote the paper : "The pipe consists of two plates, each the full length of the pipe and each bent to a semicircle. The edges are burred or beaded to the shape of a dovetail, and are inserted in the open jaws of heavy longitudinal bars, which are subsequently closed cold on the edges of the plates, thus forming longitudinal dovetail joints. The pipes when *in situ* were jointed by a simple sleeve joint, the ring being 8 inches wide and ½ inch larger diameter than the pipes bulging out to clear the lock bars.

Fig. 30 is a section of the locking bar open and closed. In Fig 30a is a section showing the sleeve joint enlarged, which is leaded, while Fig. 30b is an elevation and two sections of the joint and pipe. Fig. 30c is a section of the pipe way, showing the pipe as covered up by earth.

**(67)** **The largest of the service reservoirs, the one at Balla Bulling, is formed of concrete reinforced by barbed wire, and provided with bituminous joints right round near the toe of slope. This is to allow for expansion and contraction, as no concrete without such relief can stand the alternations of temperature in that climate without cracking.**





FIGS. 31, 31a.—Balla Bulling Reservoir.

In Figs. 31 and 31a we have a part section and plan of this reservoir. The whole of this immense work cost £2,660,000.

(68) Note has already been made of embankments constructed without puddle walls. An instructive example is given of American practice in the reservoir embankment of a settling reservoir at Belmont, Philadelphia, given in Fig. 32.

The clay puddle is external, 18 inches deep, covered with 6 inches of concrete. The depth is only 25 feet.

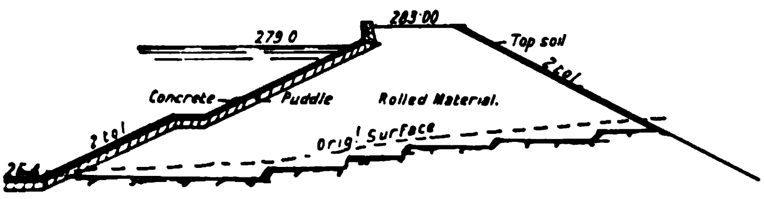
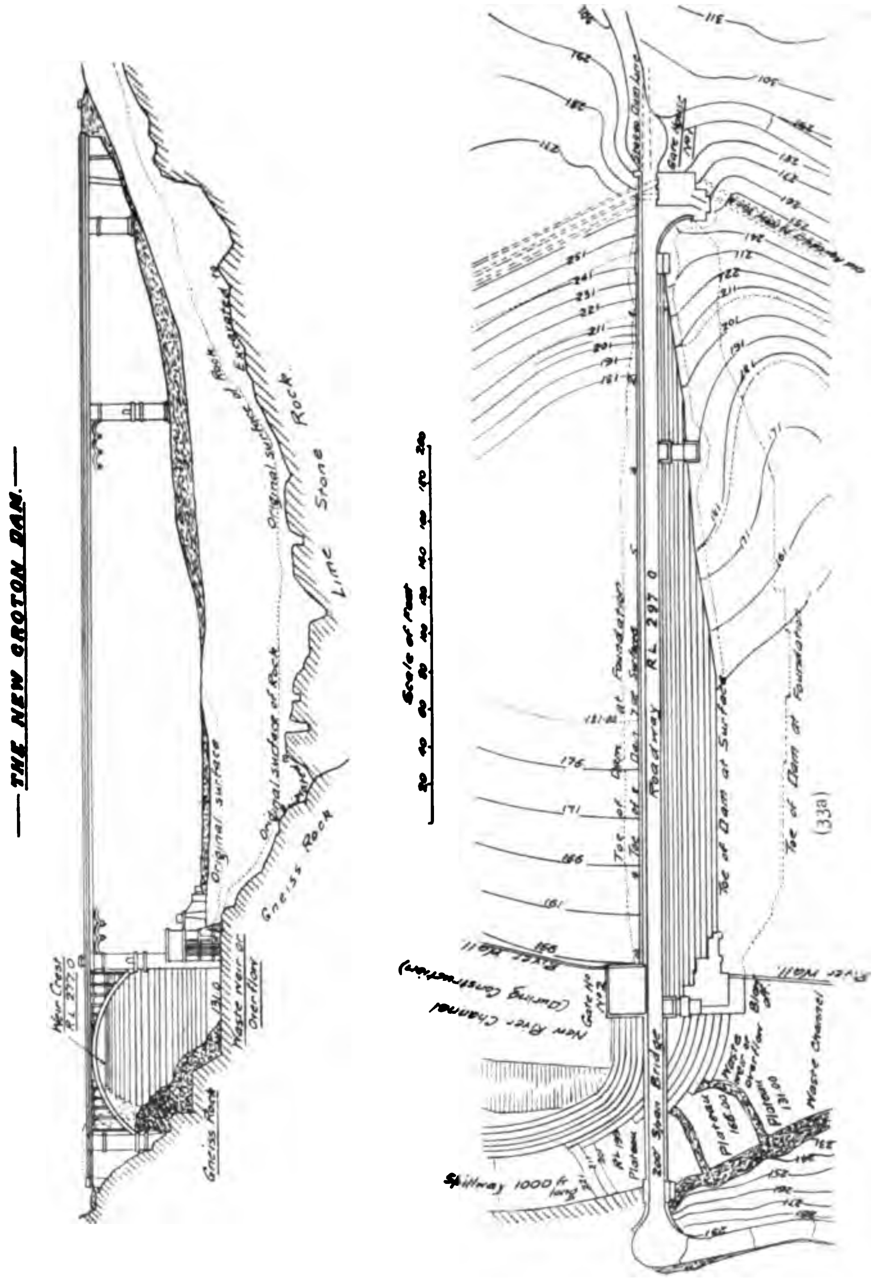


FIG. 32.—Belmont Dam.

(69) The new Croton Dam for the water supply of New York is a recently constructed work of great magnitude. The works consist of a dam carrying



FIGS. 33, 33a.

a roadway 20 ft. wide on the crest and at right angles to it the waste weir is set, forming one side of the reservoir. By this remarkable arrangement one flank of the river valley is utilised for the weir and the escape channel which is walled off from the actual river channel below the weir. The original water way down stream is filled up to a high level and utilised as a park or recreation ground.

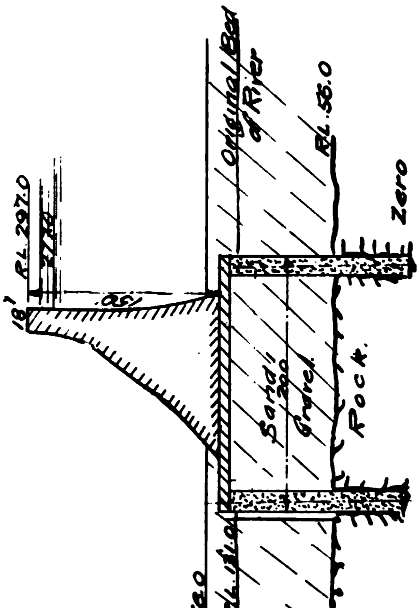


Fig. 33c.—Alternative Section.

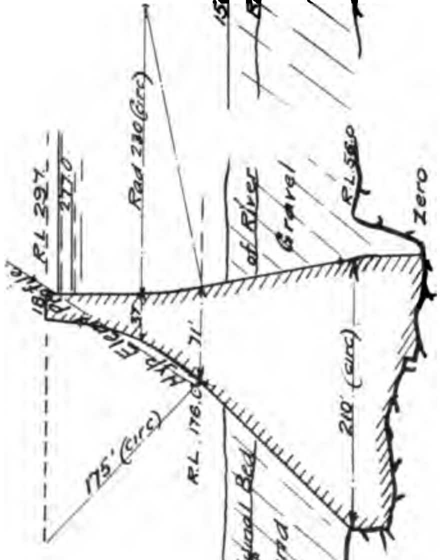


Fig. 33b.—New Croton Dam. Section.

The dam portion of the work is 1168 feet and the river 1000 feet in length. The plan and elevation of the work, for which the author is indebted to the *Engineer*, are given in Figs. 33 and 33a the section of the dam in Fig. 33b. The section of the dam is remarkable for its immense size and the fact that the greater portion of its depth is buried out of sight below the bed of the reservoir. The maximum unit pressure is 20 tons. Owing to a fault in the rock foundation, it was deemed essential to carry the solid masonry down to this immense depth, through 150 feet of sand and gravel. The rock eventually met with formed probably the bed of the ancient glacial river and the material dug out was deposited in that remote age. This material although porous cannot but be deemed a most excellent foundation in a masonry superstructure and certainly superior to made earth provided percolation was stopped. This, it is maintained, could have been effected with perfect safety by excavating a trench up stream through the material and well into the rock and filling the same up afterwards with cement concrete. If deemed insufficient for further security a second curtain could be constructed down stream of the dam but there are objections to this. The arrangement is shown in Fig. 33c.

(**70**) The sectional area of the dam as shown in this figure is estimated at 8,000 superficial feet, the value of $Ap$ is therefore 8,000 × $2\frac{1}{4}$ = 18,000; that of the water pressure, or $CH$, is 75 × 150 = 11,250. Hence the frictional stability of the sand foundation is assured as regards weight, but the length of creep is deficient, it should be 12 $H$ or 150 × 12 = 1,800 feet; this could be secured by a clay floor extending upstream for 1,600 feet, in which case, according to the theory expounded in Chap. VI., the foundation would not be washed out, even were the curtain walls not provided. With this provision, the section, as shown in Fig. 33c, is undoubtedly perfectly secure, and would further be watertight against percolation. The original plan for this work was an earthen dam with a core wall excavated into the rock. Owing, however, to suspicion as to the nature of the rock this plan was abandoned, and a solid masonry weir adopted.

(**71**) If the core wall has been properly carried down well into the solid rock there is no reason why an earthen dam should not have answered, provided the original river-bed was left undisturbed and the embankment built on top of it. There are many instances of reservoir embankments thus constructed overlying porous material, which latter is rendered watertight by means of a core trench wall of puddle or concrete. To carry the base of a masonry dam right down into the solid rock appears a most unnecessarily expensive arrangement. In the Fig. 33c the horizontal platform should not overlap the vertical core walls.

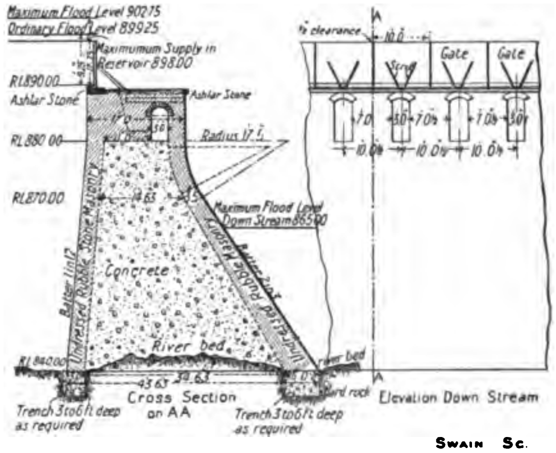


Fig. 34.—Section of Dhukwa Dam.

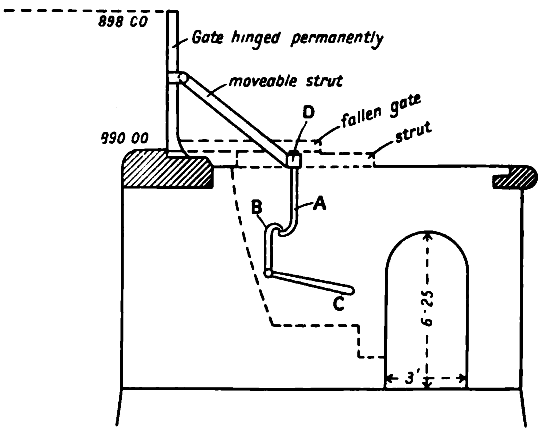


Fig. 34a.—Automatic Water Gates.

(**72**) Through the courtesy of the *Engineer* in lending the requisite blocks the following insertion of a brief account of the new Dhukwa Weir over the Betwa river in Bundelkhand, India, has been rendered possible, while this book is actually in the press. The arrangements adopted in this work of dropping and raising the collapsible weir shutters, present features of considerable interest, as they do not involve the necessity of an over-bridge, They are fully illustrated in Figs. 34 and 34a. In the latter figure it will be seen that a rod marked A, provided with an hooked end is attached to the base of the moveable struts of each shutter, this end is engaged by the similarly curved end of an arm of the lever B C.

The lever handles C are all situated in recesses connected with a tunnel which runs longitudinally throughout the weir. By pressing the handle C the rods A and B are disengaged and the shutter will fall. The gates can thus be let go either by an attendant or by the handles being connected with a cable worked from each end of the weir.

The raising of the shutters will be effected by means of a crane, running on rails on the weir crest itself. The crane as it moves along will be sheltered from the rush of water over the weir by being situated well in rear of the shutters it has already raised in its advance. The connection of each strut with the lever B C will simultaneously be effected by a man operating from the tunnel below.

The weir shutters are 8 feet high.

**(73)** The section of the weir (Fig. 34) is a great improvement on that of the old Betwa or Parichha Weir, the heavy profile of which is by no means to be commended. This latter has not been illustrated in this work, although the Head Regulator of the Betwa Canal, an adjunct to the old weir has been reviewed in Chap. VIII.

Like many American examples the weir will impound water to a level well raised above the shoulders of the flanks of the river channel proper. The work will thus consist of a central overfall weir crossing the defined river channel, its continuation on either flank being an insubmergible earthen dam provided with a masonry core wall. The two earth dam portions are about 1,000 feet long each and some 35 feet high, while the central masonry weir is about 60 feet high to crest of the shutters. This arrangement is exactly the reverse of that obtaining in the Minidoka Head Works (Fig. 16), or of the Roosevelt dam (Fig. 17 of this chapter), where the waste weirs are on the flanks, the central part being an insubmergible dam.

The flood discharge of the Betwa River is estimated at 800,000 cusecs, the available impounded supply will be 86,000 acre-feet. This reservoir will be tapped by three large body sluices, 8 × 6½ and 8 × 10, worked by Stoney Gates, and will act as a feeder to the lower old Betwa Reservoir and to the canal which takes off from it. How these sluices are to be worked, whether from the tunnel or from top of special piers built over each on the weir crest, is not stated.

**(74)** The obviously weak points in the design of the regulation apparatus consist in the multiplicity of the gates, which not being automatically collapsible have to be lowered as well as raised. The weir crest is no less than 4,000 feet in length, consequently 400 gates will have to be manipulated. Arrangements are contemplated whereby they can be lowered in batches, but the raising of the shutters will necessarily be a very slow operation. In addition to this, considerable leakage is bound to occur into the subway by means of the slots through which the vertical arm *A* attached to the longitudinal *D* must move back and forwards as the shutter is raised or lowered. These will doubtless be covered by stanching strips, but still under a possible head of 13 feet of water leakage is bound to occur. The disadvantages inherent in this system are considered as so formidable as to render even its trial a matter of doubt.

(75) Two alternative systems presenting advantages from some points of view have already been illustrated in Figs. 18 and 11. In the former, that of the Folsam canal weir, a hinged trussed shutter, 150 feet long and 5 feet deep, is raised and lowered by means of a series of hydraulic jacks, the plungers of which act as the supporting struts. This system could well be adopted for the Dhukwa weir, with, or even without the necessity of piers; the matter of expense for power installation being the only question to be considered. This would necessarily be very considerable, owing to the great length of the weir crest.

The second alternative system is that illustrated in Fig. 11, viz., of inclined roller gates working up from below the crest on the inner face of the weir. In this system, however, piers will be necessary and the spans could not well be very large, as the pressure instead of being distributed generally along the weir crest will be concentrated at the piers, so that unless buttresses were provided below the piers the spans could not well exceed 20 or 25 feet, which would not be sufficiently wide for the passage of a torrent in violent flood, as the Betwa river. With this provision, however, and that of a subway instead of an overbridge, the system is quite practicable for spans of 50 or 100 feet.

(76) The author believes that a modification of the principle exemplified in the case of the Schweinfurt weir in Germany, which is illustrated in Wilson's "Irrigation Engineering," page 184, will be found capable of satisfying the contradictory requirements of economy, simplicity, and rapidity of action in the regulation of this weir, and be found superior to either of the above systems. In the Schweinfurt weir, the raising of the water level above the crest is effected by the adoption of a hollow steel cylinder internally braced, of a length of 150 feet, which lies on the weir crest between the two abutments. In flood-time this cylinder is raised clear of the weir and of the flood-water by being revolved, and it thus mounts a steep inclined shoulder, cut in the sides of the two abutments by means of encircling chains, one end of each of which is fixed and the other wound up on the drum of a motor winch. The cylinder is thus rolled bodily off the support of the weir clear of the flood, and is similarly lowered when closure is required.

(77) The development of this principle, shown in Fig. 35, is to effect the required regulation, not by lifting the cylinders off the weir crest as was previously done, but by retaining them on it, the crest being also inclined and built on an incline of one in three. Regulation is then effected by rolling the cylinder up and down this inclined plane. The cylinders are never intended to be lifted clear of the support of the weir crest, and consequently are entirely free from the great bending stress induced were they

supported only at each extremity, as in the example already noted. Further, the power required to move them is very slight, as the inclined plane is of moderate slope, and the friction is a rolling friction. They can then be made of considerable length and be formed of such economical a material as reinforced concrete. The cylinders will be revolved by steel ropes or chains encircling each end, which will run in a recess cut in the side of the piers. One end of the chains will be fixed to a hook at the summit of the run, while the other will be carried round the cylinder on to a pulley and thence to the drum of a hand winch fixed on the top of the pier. The cylinders will fall back down the inclined crest by their own weight, when the supporting bands or chains are unwound.

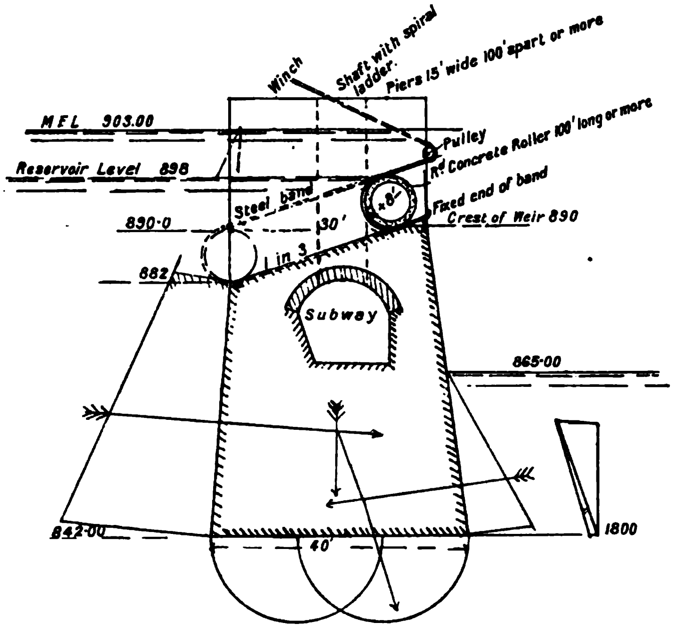


FIG. 35.—Alternative Design of Dhukwa Weir.

For the prevention of silt deposit on the inclined crest it is proposed to admit water through corrugations in the surface underneath the rollers, so that the only quite watertight connection will be at the summit of the run. No overbridge is needed, the subway system with the addition of vertical shafts at the piers being maintained.

In the diagram the summit of the trapezium of pressure should have been placed at R.L. 890, not 882. This correction would bring the centre of pressure just at the middle third of the base.

# CHAPTER XIV.

## SCREW GEAR FOR TANK SLUICES, AND ROLLER GATES.

(1) IT is proposed in this chapter to give a short account of the best type of screw lifting gear suitable for tank sluices, as also of different types of rollers in use in vertical draw sluice gates.

Figs. 1 to 4 and Fig. 7 are rough sketches illustrative of the principle of different systems of screw lifting gear. In Fig. 1 the motive power is applied to a female screw cut in the base of the handle itself. On revolving the handle the solid screw rising, lifts the gate, but there is no means of forcing the gate down, unless its dead weight is sufficient for the purpose. The author has actually seen this extremely primitive gear applied to wooden sluices of Government tanks in Burma, and was privileged to witness the operation of lowering the gate against a head of water.

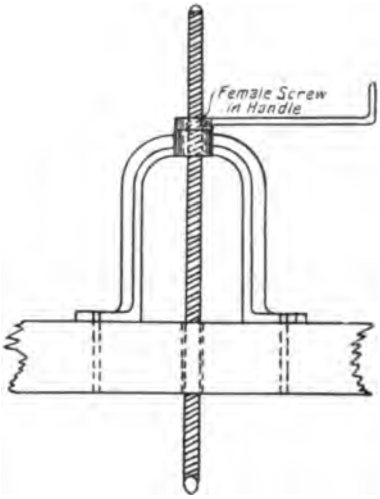


FIG. 1.

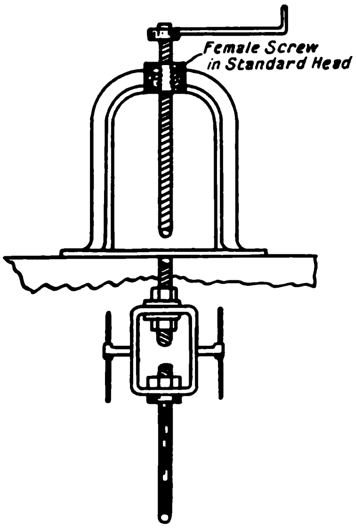


FIG. 2.

The procedure adopted was for one cultivator to stand and jump on the gate while his companion belaboured the top of the screw rod with a large stone, the handle having been previously run up to the top. The screw rod, it may be added, was fixed to the gate at its lower extremity and so could not itself revolve.

(2) In Fig. 2 the motive power is applied to the screw rod, the female screw being cut in the head of the standard. In this case the screw rod

itself revolves, and consequently has to be provided with a dog chain swivel joint either at the gate fastening, or preferably as near the screw head as possible, thus reducing the torsional stress on the rod. The handle rises and falls with the rod, to the extremity of which it is keyed.

This type is a great advance on the last, and is quite suitable for most plugs which are only lifted 2 feet at the outside. The swivel should be prevented from turning by two arms which slide up and down guide rods. This is shown roughly in the sketch.

(3) Figs. 3, 4 and 5 represent the ordinarily adopted lifting gear, in which the female screw is revolved. The solid screw rod connected with the gate passes through the former and rises above the platform. This arrangement is on the same principle as Fig. 1, only the female screw itself is prevented from rising by the use of a thrust plate which either works in an annular groove cast in the brass screw head, as in Fig. 3, or itself is provided with a holding down, as well as a base plate, which embrace a circular collar cast or screwed on to the head piece as in Fig. 4.

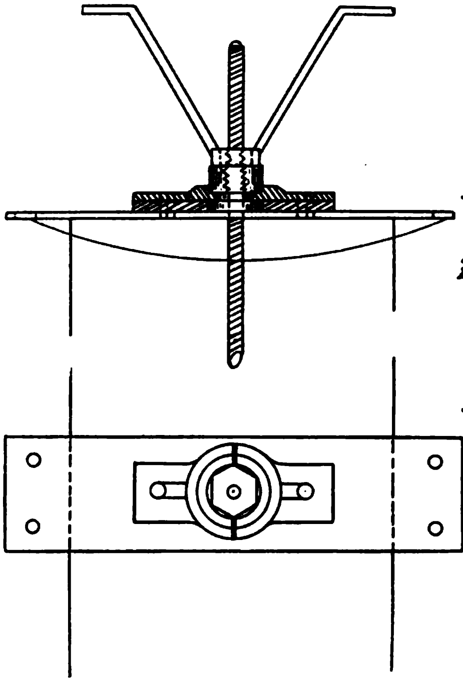


Fig. 3.

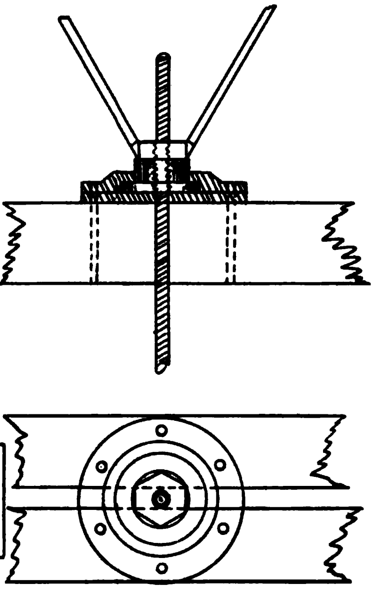


Fig. 4.

In Fig. 3 the steel or iron plate is in two halves, and each half is provided with a slot as shown in plan. This enables the two halves of the thrust plate to be disengaged and removed by slackening the fastening nuts and pushing the plates backwards, the bolts traversing the slot. The author has seen several of these employed in lifting tank sluices, but they proved very hard to work. In Fig. 4 the holding down upper plate is in one piece, the brass female screw being provided in this case with a collar at its base. Fig. 5 is taken from the "Madras Manual," and exemplifies the usual Madras practice, where screws are very commonly employed. In this case the thrust plate also is in one piece and is slipped over the projecting shoulder of the brass screw head and bolted in position. If this thrust plate is truly cast and planed it should be as good as, if not preferable to, the form illustrated in Fig. 3.

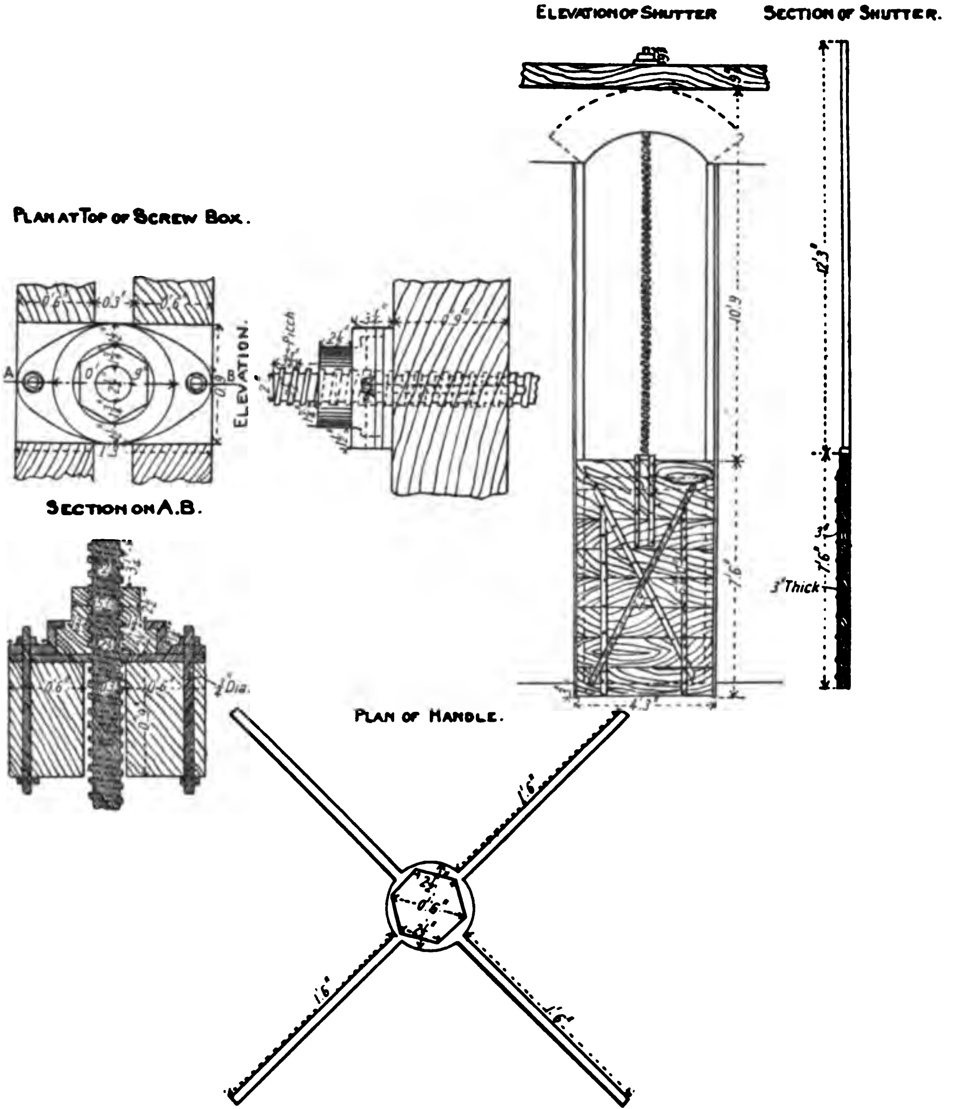


FIG. 5.

(4) Fig. 6 represents another development of the same principle, and is used in Madras for lifting a series of gates one above another, which close the larger openings of Undersluices or Head Regulators. As already

mentioned in previous chapters, this style of lifting gear for such purposes is now completely obsolete. The introduction of anti-friction rollers enables gates when released to fall into position by their own weight without requiring the application of any power to force them down against a head of water. Hence they can be lifted by a travelling winch, an arrangement much more economical than the lines of long expensive screw rods shown in Fig. 6. In this gear the female screw is formed at the end of a long pipe, at the top of which pipe is a solid head to which the power handle is applied and which is held fixed, by a thrust plate. As the pipe is revolved to raise the gate, the solid rod attached to the latter is drawn up inside the pipe, being thus protected from dust or water.

As screws in future are never likely to be employed except for very short lifts, such as those of small deep reservoir sluices, of which the outside lift is 3 feet, it is quite clear that the principle of the motive power being applied to the short hollow screw, thus involving a long solid screw rod subjected to compressional and torsional stress, is not at all economical.

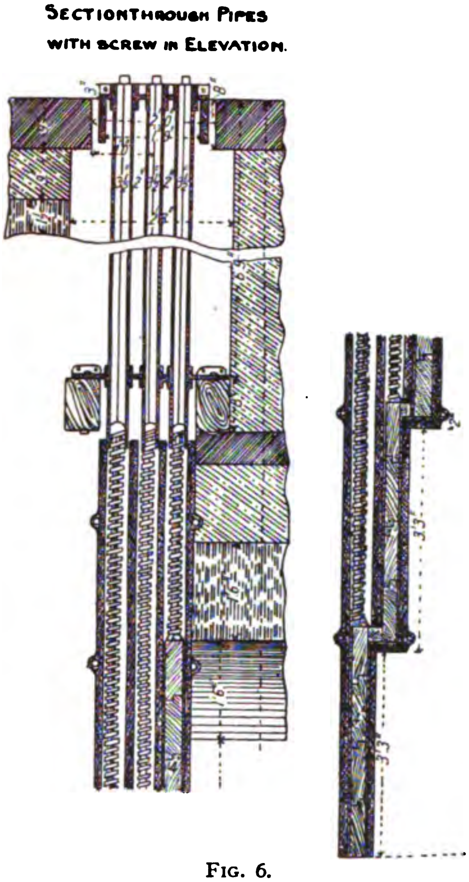


FIG. 6.

(5) The arrangement illustrated in Fig. 7 is undoubtedly far superior to the former. In this the power is applied to the male screw, which can be quite short, a little over the lift in length. This should be of solid cast steel. As shown in the illustration, the thrust collar and plates are situated at the rod head, and consist of three plates, the upper, the distance plate, and the base plate. The screw rod is threaded through the base plate, while the upper plates are superimposed, and the whole bolted through as shown on plan (Fig. 7). The upper and lower plates should be of brass, the middle distance plate, which is subjected to no strain or friction, being of planed iron, or else the middle and top plate could be combined in one piece of brass, as in Fig. 4. The solid screw rod passes into a pipe in the head of which is the female screw. This should be of brass, and in some cases the whole pipe is made of brass; but this seems a needless extravagance. The pipe head is held rigid by two arms running in guide slots cut in the frame of the standard. Thus the torsional stress is at

once absorbed and the position of the ends of the arms, one of which can be made to project beyond the frame, indicate by means of a pointer on a graduated scale the exact height at which the gate stands above its sill. The pipe is rigidly connected with the gate, and from its ring section is clearly much better suited to withstand compression than a solid rod of greater weight of metal. It can also be made of ordinary gas or water piping.

(6) This system for short lifts is by far the best of any other illustrated in these figures. It is remarkable that it is not given in the "Madras Manual," all the screw lifting gear in which are of the other types, in which the motive power is applied to the female screw. The only tank sluice fitted with this gear seen by the author was that in Kalawewa Tank, in Ceylon. In this a hand wheel turned a worm, or archimedean screw, which engaged a bevel wheel keyed on to the top of the shaft. The sluice gate in this case was 3 feet square under a head of 25 feet, and yet the gate was manipulated with the greatest ease by one man.

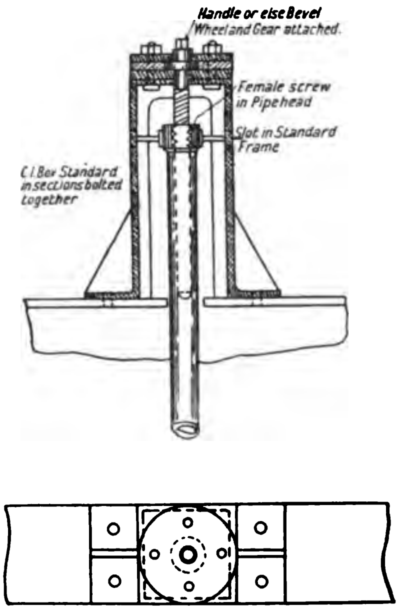

Handle or else Bevel Wheel and Gear attached.
Female screw in Pipehead
Slot in Standard Frame
C.I. Box Standard in sections bolted together

FIG. 7.

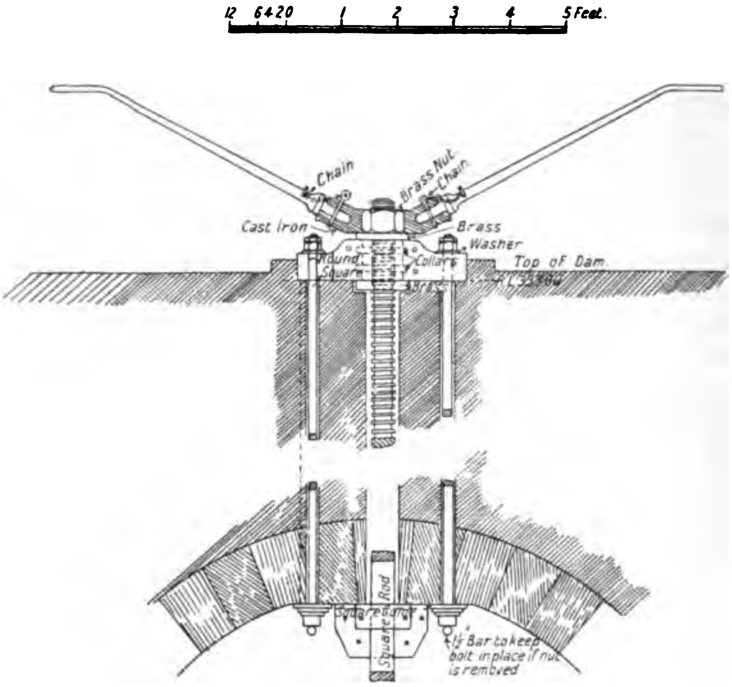
(7) The screw heads, or standards, can be supported on wooden beams, or a cast iron fish bellied girder with double webs. Two rolled beams connected by a plate bolted to their upper flanges would suit equally well, the lifting rod passing between their webs.

(8) A very instructive example is given in Figs. 8, 8a and 8b of the screw lifting gear of the Bhatgarh Dam undersluices, a section of which is in Fig. 18, Chap. II., and for which we are indebted to "Irrigation in India." These sluice gates have a lift of 8 feet under a mean head of 80 feet, the area of the sluice ways being 8 feet by 4 feet; the pressure on each is therefore $whA = \frac{1}{36} \times 80 \times 4 \times 8 = 71$ tons. The screw rod is 5½ inches diameter for the screwed upper length, the remainder being 5½ inches square. These heavy, expensive forged rods are about 87 feet in total length, though made up in 12½ feet lengths, and are supported at intervals against buckling by passing through square holes in brackets projecting from the rear face of the dam.

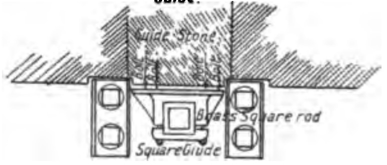
The motive power is, as usual, applied to the brass female screw, which is provided outside with two collars which work in corresponding grooves in a thrust box. This latter, which is of cast iron, is in two pieces, bolted together horizontally and further provided with the usual vertical holding down bolts (*vide* plan of lifting nut collar, etc., in Fig. 8). The squared part of the rod commences at the termination of the screwed upper part, which has a run of 8 feet, and this passes through a square brass guide which prevents the rod revolving, and relieves it of any torsional strain below this point. The screw lifting rod rises through the capstan nut, and when the gate is fully open must stick out some 9 feet above the platform level on top of the dam.

BHATGARH RESERVOIR

LIFTING GEAR TO UNDER SLUICES.



Plan, looking upwards, of holding down Plates and Square Guide.



(8a)

FIGS. 8, 8a.

(9) This manifestly unsuitable arrangement could easily be avoided by adopting the general design of Fig. 6, viz., of applying the motive power to the male screw, which latter being provided with a collar working in a thrust box, is prevented from vertical movement but not from revolving. The female screw would be placed as in Fig. 6, in the head of a long cast iron pipe, which is fastened to the gate, and which is prevented from revolving by attached arms, the extremities of which slide in vertical grooves,

and is, of course, free to move vertically in either sense. It is quite evident that this arrangement would be much less expensive and equally effective as that portrayed in Fig. 8.

A further improvement would be to apply bevel gearing to the screw head in lieu of the capstan, which has to be worked by several men. A bevel spur wheel could be fixed horizontally on the rod head, worked by one or two bevelled pinions turned by wheel handles revolving vertically.

(**10**) The principle of the application of free rollers and roller wheels fixed on axles to draw gates, together with the devices employed in rendering the gate watertight, will now be briefly noticed.

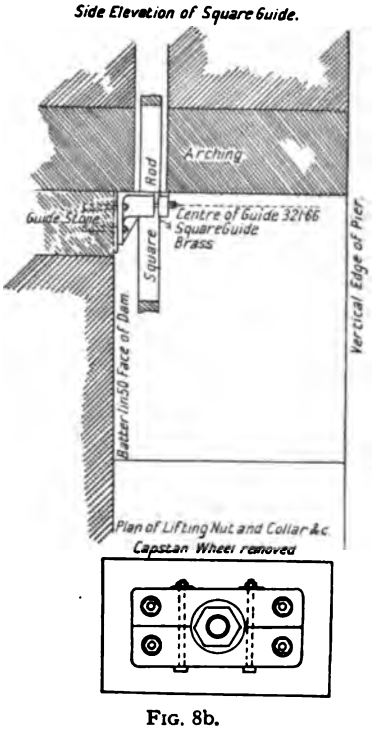


FIG. 8b.

When a gate or shutter slides in a groove, the pressure of the water forces it against the inner side of the vertical grooves, as also against the lintel at top and the sill at the bottom, thus effectually preventing any leakage on all four sides of the shutter. The resistance of the friction of the end side of the gate against that of the grooves is very considerable, and as the parts in contact cannot be lubricated, and the gate often remains unmoved for a long time, the parts become rusty, and the friction is thus increased greatly in excess of what would normally be the case. Now the coefficient of friction of smooth metal surfaces in contact unlubricated is about ·2. Thus if a gate were subjected to a horizontal water pressure of 1,000 lbs., it could be lifted, neglecting its own weight, by a force of 1,000 × ·2 = 200 lbs. On the other hand, when the surfaces are not smooth, the coefficient of friction may possibly become as great as unity. Hence a gate without rollers requires power, not only to lift it, but to force it down, as its own submerged weight is insufficient to overcome the friction induced by the pressure of the water. This is the reason why screw power has hitherto been so largely used as lifting apparatus.

(**11**) By the simple device of mounting the gate on wheels with axle bearings, the friction is greatly diminished, so that the requisite lifting power is considerably reduced, and none is required to lower the gate, that being effected by its own weight.

The combined axle and roller friction now induced, which is similar to that of a railway truck, can be taken as one-eightieth of the water pressure plus the weight of the gate itself in the process of lifting. Thus the net lifting power required, supposing the water pressure to be 1,000 lbs., would be $\frac{1{,}000}{80} = (12\frac{1}{2} + W)$ lbs.

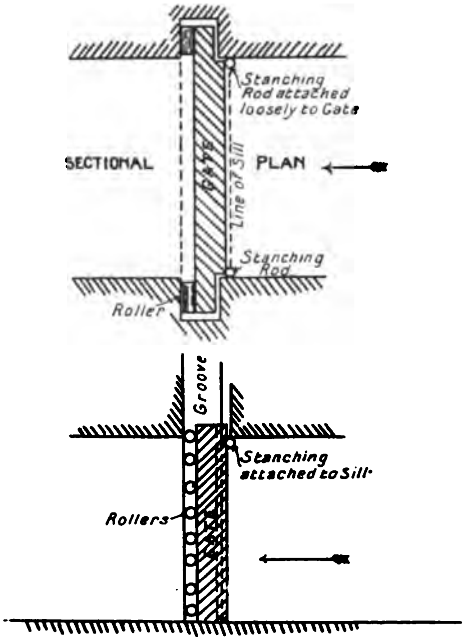
If the axle bearings are not lubricated the coefficient of friction will naturally be much greater. Within certain limits a roller wheel of large diameter would give better results than one of smaller size. The rolling friction induced between the wheel rim and the surface would be less in the former case, and in addition the leverage of the greater wheel radius acts in favour of the motive power.

**(12)** In the older patterns of roller gate, as the Narora and Chenab Undersluice gates, illustrated in "Irrigation in India," the device of the use of stanching rods to prevent leakage, which must occur owing to the gate edge not being in contact with the side of the groove, appears not to have been known; any way, there is no example of the use of stanching rods given in the work above quoted. One arrangement made use of was to build the shoulder of the groove on a slight slant, so that a plate projecting from the gate came in close contact with this shoulder when the gate was down, thus producing a watertight joint. On the gate being lifted the stanching plate gradually worked clear of contact. In another, on the same principle, the bed of the roller was slightly cut into the groove at the end of the run, so as to bring the side of the gate when quite lowered in close contact with that of the groove. These devices may now be considered quite obsolete, as stanching rods are so easy of application either outside or inside the groove, or horizontally on a lintel or over the intervals between two or more parallel gates, as will be illustrated later.

**(13)** We have hitherto considered the combined axle and rolling friction which is induced by small wheels fixed on axles running in bearings bolted to the gate. When, however, the pressure is very considerable, due to a great head of water or the exceptional dimensions of the opening, another arrangement, viz., that of free rollers, is made use of. These rollers run on axles, but there is no pressure induced on the latter; the axles are only used to keep the rollers the proper distance apart, and are fixed in a frame. This roller cradle is suspended in the groove unattached to the gate. As the side of the gate bears against the rollers, when the former is moved the rollers revolve, and the whole frame of rollers rises and also falls with the gate. The friction induced is pure roller friction, which, between smooth surfaces, is something very small indeed, so much so that the coefficient of this friction can be entirely ignored, the lifting power being only the weight of the gate plus the friction of the lifting apparatus. Again, under suitable conditions, the gate can be counterpoised by weights hanging from overhead pulleys, and in such cases gates of the largest size can be manipulated by one man. These free rollers are termed "Stoney's Patent Anti-friction Rollers."

**(14)** The diagrams in Fig. 9 represent a sluice gate fitted with free rollers in sectional plan and elevation. It will be seen that the frame of the gate on plan just clears the sluice wall, and projects beyond the groove. These spaces are closed by stanching rods. These are simple round rods of about 2 inches diameter, which are fastened to the gate, but have lateral play, so that when the gate is under pressure they are forced into the corner, effectually closing the aperture. On the gate being raised they are carried up with it, still bearing against the side of the gate and the sluice wall; the friction thus induced is, however, quite trifling.

The cross-section shows that there must exist a clearance space between the fore lintel of the sluice and the gate, which latter must run up clear of the former. This space is closed by a horizontal stanching rod, which, however, is attached to the lintel itself, the gate scraping past it as it moves up or down. The free rollers are shown in the groove. Supposing instead of free rollers, wheels on axles running in bearings attached to the gate a pair on each side were substituted, the general arrangement could be precisely similar.



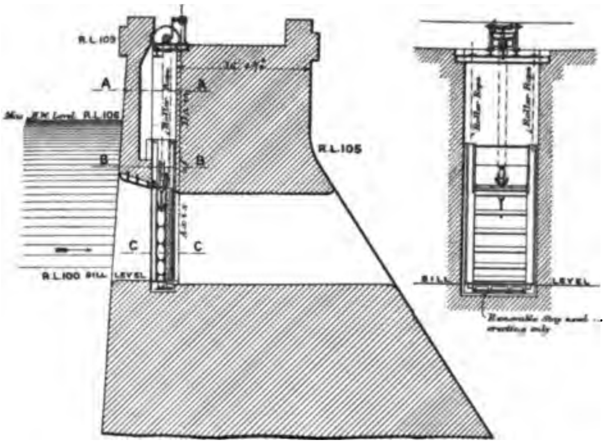
CROSS SECTION.

FIG. 9.

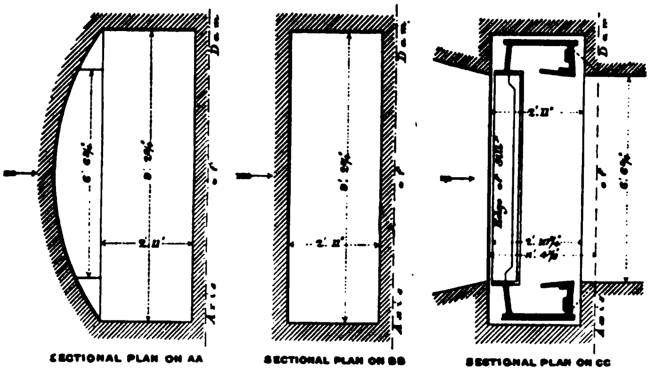
(15) In Fig. 10, which is a replica of that in Vol. CLII., "Minutes of the Proceedings of the Institute of Civil Engineers," the details of the roller gates used in the sluices of the Assuan Dam, viz., those at R.L. 100·00, are given. The following is the description by Mr. F. W. Scott Stokes, M.Inst.C.E., the author of the paper on the sluices and lock gates of the Assuan Dam.

(*Sluices at R.L.* 96 *and R.L.* 100.)

"The culverts are 2 metres (6 feet 6½ inches) wide by 3·5 metres (11 feet 6 inches) high. The entrance to the culvert is well mouthed and is roofed over by a casting curved on the dam face, and flat at the place where it joins the sluice lintel. The grooves, lintel, and sill of the sluice are of cast iron arranged with machined faces, and put together with turned bolts. The sluice is built up of a steel plate skin with rolled strengthening girders at the back, framed into cast iron beams on each side, which form the roller paths. Adjustable bars are fitted on each side of the face, to reduce the leakage as wear takes place owing to the cutting action of the silt in the water. On the top of the sluice is also fitted an adjustable bar, which shuts down on the lintel at the same moment that the bottom of the skin lands on the sill, and thus makes a watertight joint. The rollers are arranged in cradles formed of flat bars, and are hung in position on each side of the gates by steel wire ropes. The crab has two speeds of lift, namely, for small adjustments by one man, or for quick working by four. An automatic self-sustaining gear is provided, so that the sluice (which is not counterbalanced in any way) cannot run down, the handles having to be turned when lowering in order to release the gear. The barrel shaft is worked by means of a worm wheel, and a worm fitted with a ball thrust bearing. All the bearings and pulleys are fitted with screw-down grease lubricators; this is the more advisable owing to the sand and dust."



CROSS SECTION OF DAM, SHOWING SLUICES AT R.L.100.



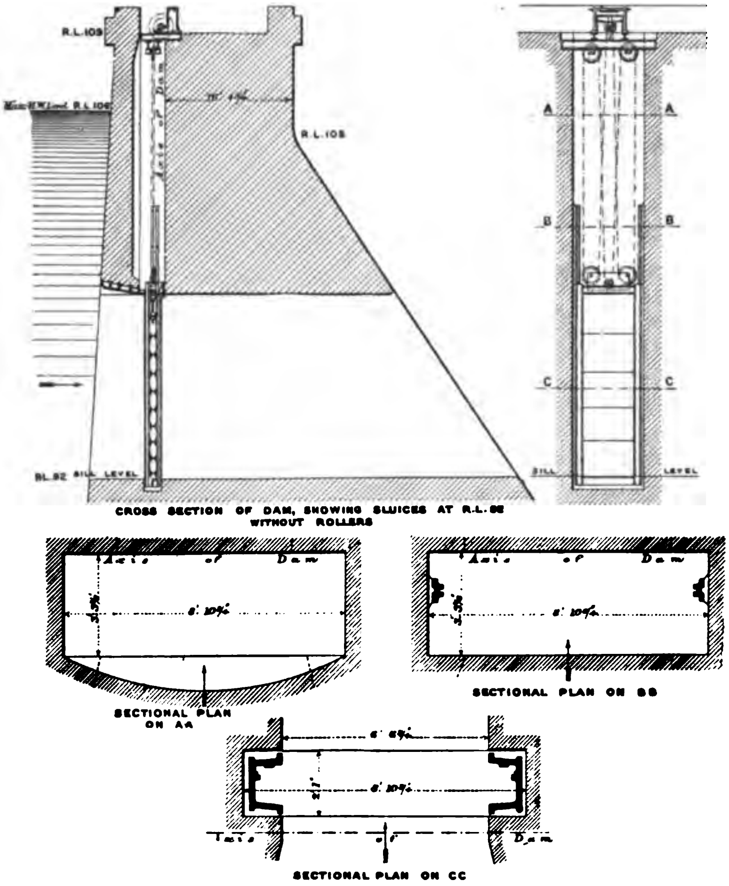
SECTIONAL PLAN ON AA SECTIONAL PLAN ON BB SECTIONAL PLAN ON CC

FIG. 10.—Assuan Dam.

(*Sluices with Rollers at R.L.* 92.)

(16) "These twenty-five sluices are 7 metres (23 feet) high by 2 metres (6 feet 6¾ inches) wide, and are for the purpose of regulation when the fifty without rollers have been lowered, and before the water rises high enough for those at R.L. 96 and R.L. 100 to give the required discharge.

"As the head above sill level is 14 metres (45 feet 11 inches), the rollers and other parts have to be of much more ample proportions, and the grooves built into the masonry of considerably larger size than those of the sluices of R.L. 100 and R.L. 96. The action of the water passing through the culvert at a high velocity has also to be avoided; hence the rollers are kept well back from the face, and an inclined shield plate is provided to protect them from a direct flow. A whirlpool motion is set up in the grooves, but it is not of sufficient force to disturb the rollers hanging below the bottom of the gate when raised. The grooves are built of four sections weighing about 2 tons each. They are bolted together with turned bolts, and are arranged with projections on each side to key into the masonry and prevent the leakage of water round the back. The sluice is built of heavy, specially rolled steel joists, with steel skin plates and cast iron roller path guiders on each side.



CROSS SECTION OF DAM, SHOWING SLUICES AT R.L. 92 WITHOUT ROLLERS

SECTIONAL PLAN ON AA

SECTIONAL PLAN ON BB

SECTIONAL PLAN ON CC

FIG. 11.—Assuan Dam.

"A vertical rod, which is contained in a groove bolted to the skin on each side of the face, is pressed by the water into the corner formed by the face of the sluice and the face of the fixed frame, and thus making a watertight joint."

(*Sluices at R.L.* 87·50.)

**(17)** "It was originally intended to have the lowest sluices in the dam at R.L. 84, but it was found that R.L. 87·50 would suit the surface levels and channels better. The head of water above sill level is 18·5 metres (60 feet 8 inches), giving a pressure of about 210 tons against the sluice. The entrance to the culvert is bell-mouthed, and roofed by an arched casting. A slight drop is arranged on the down-stream side of the sluice sill, which has been found to reduce greatly the wear and tear on the culvert bottom, and to give a freer discharge under the sluice. A similar drop has been given to the other culverts. The grooves are built of four sections, bolted together and to the lintel and sill castings. Stanching rods are fitted to the sluice, as in those for R.L. 92 with rollers. The gate is built of special rolled steel joists framed into two cast roller paths. The skin is riveted to the joists in sections, which, when in position, are finally fixed together by turned bolts. The sluice gate is carried by a steel wire rope in two parts arranged round pulleys on the gate and on girders at the pit top. Forced grease lubrication is employed as in other sluices and gear. The crab is similar in design to those for the sluices at R.L. 96 and 100."

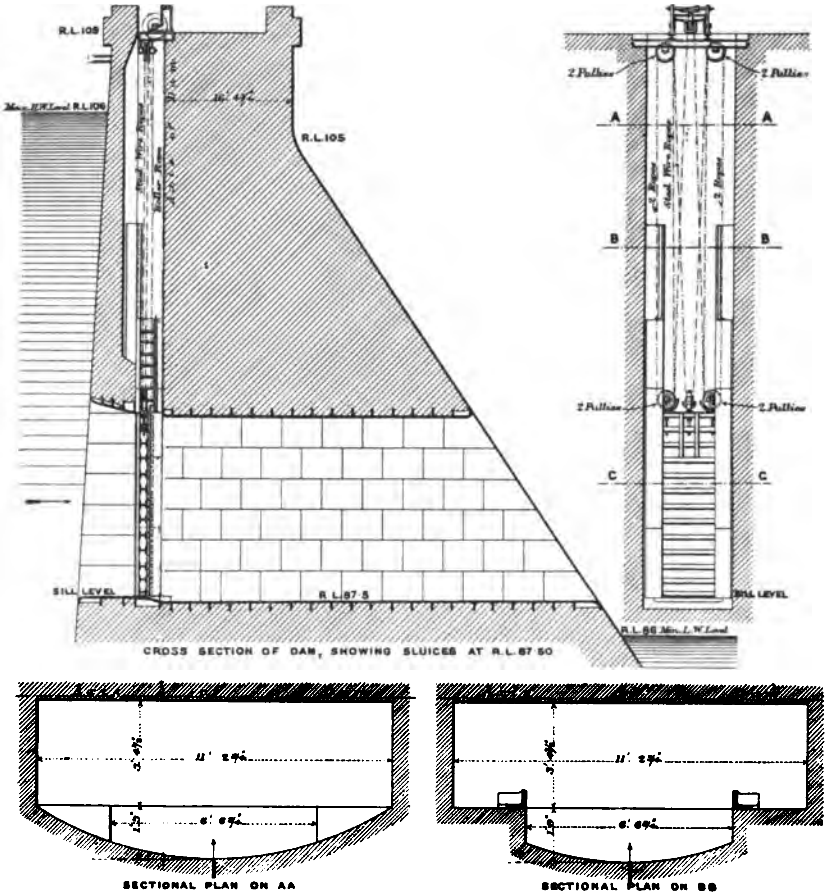


Fig. 12.—Assuan Dam.

(**18**) Fig. 13 illustrates the case of double roller gates as used for undersluices of wide span. The outer gate is generally the lower one. These gates require a watertight joint between them when both are lowered. The projection of the rollers and the thickness of the middle groove flanges places them somewhat far apart. This difficulty can be overcome by projecting the top plate of the lower gate inwards, so that a narrow flat plate will cover the space, or else a plate attached to and suitably hinged to the base of the inner upper gate, so that when the lower is raised it will lift the flap, which will still keep in contact with some projecting vertical bars inserted for the purpose, and thus either gate can be lowered to the floor. The enlarged section given in Fig. 14 will explain this.

(**19**) It will be noticed that the gates are placed with their flat sides facing. The skin of the outer gate will thus be on the outer curved side. The flap gate will have to be attached by short chains to the side so that it cannot fall much below the horizontal, in case the upper gate is moved, or else it could be raised and lowered from above by hand. This arrangement would be better than to have a pole stuffed in with straw, as used with the Chenab Undersluice gates (*vide* "Irrigation in India").

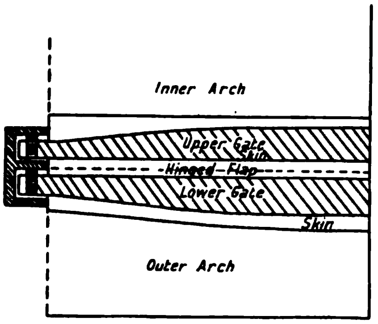


FIG. 13.

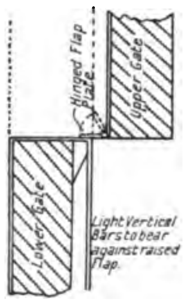


FIG. 14.

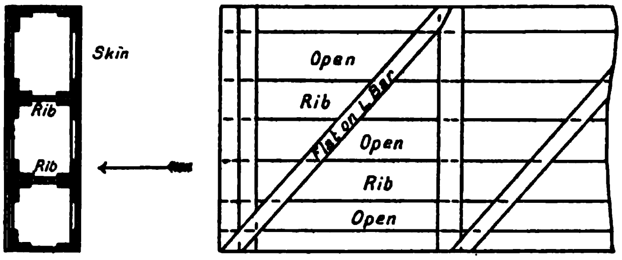
Another arrangement of the gates is to build the outer gate on plan on a curve so that the inner face is parallel with the curved outer face of the inner gate.

Roller gates are best designed with a shoulder, or else an angle iron can project to form an abutting line for the vertical stanching rods. If necessary, stanching rods can be applied inside the groove itself at the ends of the gates.

(**20**) Gates are usually built up as plate girders, but sometimes rolled beams are used for the longitudinals, as in the Assuan Dam sluices. The gate consists of the outer skin and a series of longitudinal ribs which convey the horizontal water pressure to the rollers. As the pressure is greatest at the base of the gate, the ribs should be closer together near the lower end of the gate, the spacing being gradually increased so as to cause each rib to bear approximately the same horizontal pressure. The first point to be determined is the depth of the ribs, *i.e.*, the width of the gate at the centre.

This may vary from $\frac{1}{20}$ to $\frac{1}{10}$ of the span. Representing the distance apart of the lowest ribs as $b$, and the head of water above the middle point between the ribs as $h$, and the length between the piers or span as $S$, the pressure to be supported by each rib will be $\frac{1}{2} w (h \times S \times b) = W$ in feet tons, $w$ being $\frac{1}{36}$ ton, the weight of a cubic foot of water.

This pressure $W$ is a distributed load, consequently the bending moment $M$ will be $\frac{WL}{8}$, $L$ being the distance between the centre of the roller supports, which is somewhat greater than $S$. The rib in ordinary cases will be of double T section formed of two angle irons, the upper tables of which are riveted to the skin and the lower to the web. If the skin plate on one side and the web be neglected from consideration, the sectional area of each of the flanges will consist of that of the angle iron plus the portion of plate enclosed between them, as shown shaded in Fig. 15.



*Rear Elevation of Gate.*

FIG. 15.

(**21**) The effective depth of the girder can then be considered to be not the extreme depth of the double T beam, but the distance between the centre of gravity of the angle irons or $d$, which lines fall a little below the inner side of the top tables of the angle irons (*vide* Fig. 15). Tensile and compressive stress on the top and bottom flanges will be $\frac{WL}{8d}$ tons. The safe stress on iron may be taken as 4½ tons, and that on steel as 6 tons per square inch, termed $m$. The required net sectional area of the flanges, less rivet holes, will be $a = \frac{WL}{8dm}$. If rolled beams are used, the thickness of the web will have to be taken into consideration, and the moment of inertia about the neutral axis of the beam calculated (*vide* Moles's Pocket Book, p. 130). Then $R = \frac{mI}{N}$, in which $R$ is the resistance of the beam to flexure which should equal $\frac{WL}{8}$, $m$ the safe compressive or tensile stress per square inch of the metal, $I$ the moment of inertia, and $N$ in this case $= \frac{D}{2}$, $D$ being the outside depth of the section.

(**22**) No continuous plate is required on the inside of the gate, but vertical and inclined strips of angle or flat bar at intervals, so as to enable access to be obtained for painting the parts. The area of the skin plate, as also that of these strips reduced to the equivalent of a thinner plate over the whole surface, could be included in the sectional area of the flanges, or in the value obtained for $I$. The webs of the ribs will naturally have to be strengthened by angle or T irons at intervals, as is usual in plate girders. In large single gates, where space is not confined, the ribs are generally composed of open lattice work with a good curvature to the centre similar to a bow-string girder placed on its side. The top and bottom of the gate should be plated over.

(**23**) The shearing stress on the axles of the gate roller wheels, if such be adopted, will be one-quarter the total water pressure on the whole gate, *i.e.*, if two sets of rollers be used. The position of the rollers should be fixed with reference to the pressure so as to secure the upper having the same pressure as the lower roller.

The axles, if supported on both sides of the rollers, are subjected to double shear, consequently their sectional area should be $\frac{W}{8m}$, $w$ being the total water pressure on the whole gate or $wh \times ld$, $h$ being the mean head and $ld$ the area of gate, and $m$ the safe shearing stress of the metal, viz., 5 tons for iron and 6½ tons for cast steel.

It might be further noted that the web of the ribs is subjected to shearing stress which is greatest at the ends of the gate, where it will amount to half the total water pressure on the gate; presumably it is sufficiently supported against buckling. Its effective sectional area should equal this pressure in tons divided by $m$, the safe shearing stress of the metal, generally assumed as the same or less than the compressive strength. The cover plates for joints in the skin should be of T section. These are not shown in the sketch in Fig. 15. The gate is not only subject to transverse thrust, but to vertical bending moment due to its own weight. This is met by the disposition of the bars on the rear side, as shown in the elevation, and by the skin on the face of the gate.

A good example of the construction of three-fold roller gates of the Chenab Canal Undersluices is given in Plate LI., "Irrigation in India," and of the Assiût Regulatoı Gates in Vol. CLVIII. Min. Pro. Inst.C.E.

# APPENDIX.

THE theory of the frictional resistance of sand under pressure as set forth in Chap. VI. is distinctly at variance in many important respects with the new doctrine of the impermeable floor and hydraulic gradient which was first promulgated by Mr. Beresford, C.I.E., in Vol. CLVIII., "Min. Pro. Inst. C. E.," so that a few remarks on this new theory may be considered advisable.

The frictional stability of a weir according to the theory adopted in this work, which was originally derived from the designers of the Nile Barrage apron, is dependent on the following four considerations:—(1) Effective weight; (2) Extension; (3) Impermeability; (4) The nature of the material in the river bed.

Of these the third is mainly applicable to the portion lying above the drop-wall, the permeability or otherwise of the apron below the weir not being considered a matter of vital importance, except with regard to the prevention of leakage.

In the new theory, the fourth consideration is entirely wanting, while the first is only partially applicable, and then only in the portion below the weir crest. This absence of any regard for the fourth consideration, viz., that of the nature of the river bed, which is such an important point, is of itself alone sufficient to put this theory clean of court, as no allowance whatever is made for any difference in the material of which the river bed is composed, whether it be the ebullient silt of the Nile or the heavy coarse-grained sand of the Sôn river. But there are other points equally objectionable as being quite outside the range of natural phenomena; we will refer first to the so-called hydraulic gradient.

As will be seen in the diagram this gradient is the hypothenuse of a triangle, the base of which is equal in length to that of the contour of the base of the weir including any vertical depressions, and the perpendicular is the head of water acting on the weir, the incidence of which should be placed at the commencement of the original upstream apron, not at the weir wall as in the diagram.

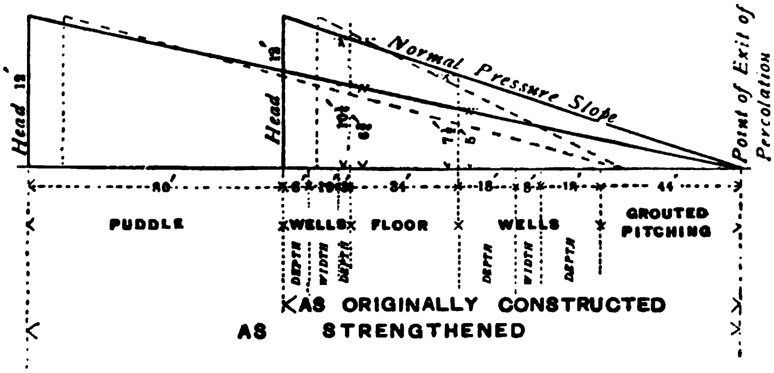
It is contended that the vertical ordinates of this triangle below the drop wall correctly represent the upward pressures against the lower apron. This view cannot be allowed to pass without challenge.

There is no reason why the upward pressure due to the head should be supposed to cease at the point of termination of the lower apron more than anywhere else. The water underneath it is practically in the same position statically as that in a submerged sluice-way with depressed roof, which has an upward pressure against the roof of the sluice-way equal to the head to which it is subjected. This upward pressure does not cease to be exerted on the roof of the sluice at its termination, and its behaviour is the same in the case of the apron. In fact, on leaving this shelter, the water immediately passes under another apron of pervious material. In this case the upward pressure is still retained, although its level beyond may rise in percolation between the interstices of the pitching or of whatever the material is composed. However this may be, it is clearly *ultra vires* to postulate the stoppage of all pressure due to the head simply because the impermeable apron happens to come to an end. The area of pressure should in fact be a trapezium, or triangle truncated at the termination of the apron.

A practical proof of the absolute unsoundness of the new theory is given by the failure of the Narora Weir itself, when the impermeable grouted pitching in the talus blew up; this could not possibly have happened were upward pressure non-existent at its termination.

DIAGRAM

TO SHOW PRESSURES ON FLOOR OF NARORA WEIR



The third point to which exception can be taken is the entire negation of any influence of the weight of the impervious superstructure, whether above or below the weir in diminishing or absorbing the head. It is true in the portion down stream of the drop-wall the apron has to be sufficiently heavy to withstand the upward pressure, but the effect of this weight on the sand substratum, which is a cardinal point in the theory of frictional stability, is entirely ignored. The head, it is true, gradually diminishes, but this progressive diminution is not in any way connected with the load imposed on the sand, but with a line arbitrarily drawn, viz., the so-called hydraulic gradient.

To push this theory to a *reductio ad absurdum*, the up-stream apron of the Nile Barrage would be equally effective were it composed of waterproof sheeting—a manifest absurdity—as there is practically no pressure on it.

Very plausible diagrams have been put forward of the correctness of the hydraulic gradient as demonstrated by the rise of water in pipes in the Narora Weir apron. These facts are, however, equally demonstrative of the correctness of the theory of frictional stability, as a few figures would easily prove.

In the repairs to this weir, the success of the extension of the up-stream platform is claimed to be due to its length and impermeability alone. Fortunately it was given a good thickness, and it was its *weight*, this discarded factor, which mainly effected the required absorption of head. In the new theory, the reduction of effective head is assumed to be entirely due to the length of creep through the sand, irrespective of its nature or of the weight above it. The incompressibility of sand which is not in a state of equilibrium owing to ebullition of water under a head of water is apparently assumed, whereas equilibrium disturbed by the pressure of water can only be met by an equal or greater pressure which acts on the particles of sand and silt, compressing them so as to prevent "piping," and absorbing pressure by the friction encountered by the water in making its way through the substratum. Length of creep is therefore a necessary factor, but must be combined with superimposed weight, otherwise it cannot exist. In the new theory these inter-dependent factors are forcibly divorced, thus violating a law of nature. In Chap. VI. the required extension down stream, *i.e.*, the length of the creep, is ensured by the use of a correct coefficient in the calculation, and further by the application of suitable ratio founded on practice. The question of extension up stream has hitherto not come up for consideration, but this length should clearly be likewise dependent on the relation of head to length in the fore slope of the Okhla Weir, which has been taken as a model for rivers of Class I. This fraction is $\frac{13}{200}$ or $\frac{1}{16}$. nearly (*vide* par. 19, Chap. VI., p. 160); thus the length should be sixteen times the head, and the sectional area be such as to give the required reduction in the effective head. For instance, in the case of Narora, suppose a reduction of head of 5 feet to be required at the weir wall, then the length of puddle and pitching up stream should be $5 \times 16 = 80$ feet, and the depth will be $5C \div 80\rho$ or $500 \div 80 = 6$ feet nearly, if $(\rho - 1) =$ unity. At the same time concentrated pressure, as that produced by the drop wall and its backing, is undoubtedly admissible, and the sudden drop in the actual hydraulic gradient, as shown in the section of the Narora Weir, clearly demonstrates the effect that concentrated weight (no doubt reasonably limited) has on the pressure curve. This calculated length tallies exactly with that actually adopted.

From all the above it is evident that, stripped of its pretensions to pose as a new scientific fact of value, this theory remains revealed as nothing more solid than a plausible phantasy of the imagination.

Impermeability is a by no means essential quality in a dam subjected to water pressure. Leakage will doubtless occur, but as long as the effective weight and extension of the superstructure are sufficient, the sand substratum will not be washed out. The Jeypore sand dam (Fig. 5, Chap. XIII., p. 335) provides an excellent proof. Here the dam is built of sand, is founded on sand, and it successfully upholds a head of no less than 44 feet of water. The Okhla and other anicuts have no impermeable horizontal barriers, and it is doubtful if the vertical masonry body walls exercise any influence whatever except hydrodynamically. The failure of the Narora Weir was due to the rear slope having been washed away, the upward pressures then naturally exceeded those shown on the diagram above.

The author is indebted to the kindness of Col. Sir Hanbury Brown in allowing use of the block which illustrates the above, and which is derived from his valuable work, "Irrigation," just issued.

The Chenab Weir, a section of which is given in "Irrigation Works of India," p. 168, has been strengthened in a similar manner to the Narora Weir by the addition of an apron up stream of the breast wall, formed of a layer of puddle, with masonry blocks superimposed, 80 feet long and 5 feet thick.

The value of $ZA\rho$ of the original down-stream section works out to 1,100. The head being 14 feet, the deficiency in the value of $ZA\rho$ will be $CH - 1{,}100$, or $100 \times 14 - 1{,}100 = 300$. Now $l$, the length of creep, should have a value of $16\,H$, or $16 \times 14 = 224$ feet. Its actual length was originally only 160 feet, leaving a deficiency of 64 feet.

The necessary length of the up-stream apron will then be 64 feet, and the necessary depth $300 \div 64 = 5$ feet nearly. If the extreme talus were excluded from consideration the deficiency of $ZA\rho$ would mount up to 410 feet, and that of $l$ to 115 feet. The up-stream apron would then become 115 feet long and $410 \div 115 = 4$ feet deep. In this $\rho$ is assumed = unity.

It is a matter of opinion whether most of the material in the apron up stream of the breast wall could not be more effectively utilised in thickening the down-stream apron, the deficiency in $l$ being met by sheet piling at the breast wall and lengthening the talus. Conversion to type D, as in the case of the Jamrao Weir, Chap. VI., it is considered, would be an improvement in this, as in all weirs of type A. The line of deep curtain blocks in the up-stream apron of the Chenab Weir are clearly superfluous, except as security against leakage.

$Z$ = sign of summation ; $A$ = sectional area ; $\rho$ = effective specific gravity.

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